APPENDIX B1
J.C. BOYLE HYDROPOWER FACILITY DAM REMOVAL DESIGN DETAILS

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1.0 INTRODUCTION

The J.C. Boyle Design appendix includes a summary of data design methodology, and other information used in the civil, hydrotechnical, and geotechnical design of the dam removal operations and structure evaluations at the J.C. Boyle Hydropower Facility.

Appendix B2 provides a summary of the hydrodynamic modeling completed to support the hydrotechnical design at J.C. Boyle, including CFD modeling and scour potential. Drawdown modeling results are included as Appendix G.

2.0 CIVIL / STRUCTURAL

2.1 MATERIAL PROPERTIES

2.1.1 GENERAL

Material properties are assessed for existing structures, in-situ soils, and construction materials that will be used for the project. Foundation conditions are discussed in the Geotechnical Data Report (VA103-640/1-2) and the Existing Conditions Assessment Report (VA103-640/1-1). Earthwork Division 31 Technical Specifications provide material specifications for the construction materials that will be used for the project. Gradation curves for the construction materials are provided on Drawings G0050 and G0051.

2.1.2 MATERIAL TYPES

Excavated materials will require engineered storage and locally sourced materials will be used in establishing temporary access or providing cover for concrete as required for the project. These materials are sourced and utilized in the following applications:

- General Fill (Type E9/E9a/E9b) and Random Fill (E10) will be sourced from the embankment, historic cofferdam, forebay, power canal, penstock, and powerhouse excavations. They will be placed in the disposal sites, tailrace and scour hole and will also be used to cover concrete. These materials are assumed to require no processing.
- Erosion Protection (Type E7a/E7b/E7c) and Bedding materials (Type E6/E8) will be used to line the final river channel and for general erosion and sediment control best management practices. These materials will require sorting by particle size or processing to create conforming materials to the gradations as shown on Drawings G0050 and G0051.
- Select Fill (Type E4) and Class II Aggregate Base (Type E11) will be used for road construction. These materials will require processing to create conforming materials to the gradations as shown on Drawings G0050 and G0051.
- Concrete Rubble (Type CR1 and CR2) will be produced from demolition of the intake, power canal, forebay and powerhouse. The demolished concrete particle size requirements are shown on Drawing G0051, but generally these materials will not require any processing.

2.1.3 EVALUATION OF EXISTING CONCRETE

Evaluation of existing concrete and reinforcing steel is not required for the J.C. Boyle demolition and removal works.

2.2 RIVER CHANNEL DESIGN

2.2.1 CHANNEL PARAMETERS

A final graded river channel has been developed for the reach through the excavated J.C. Boyle embankment that will provide long-term fish passage. The final river channel meanders and undulates through the reach with an average grade of 1% and a minimum width of 90 ft over a channel length of approximately 700 ft, and is presented on Drawings C1230 and C1232. It is expected that all embankment fill, historic cofferdam fill, and remaining sediment will be mechanically removed. The final channel will utilize the natural channel rock outcrops on the right bank, removing the need for erosion protection on this bank. A portion of the embankment and concrete cutoff wall is to remain on the left bank and will require armoring. The final river channel is designed to a 1% probable annual flood wetted perimeter. On the left bank within the embankment footprint, above the storm flood wetted perimeter, the channel slopes will be continued at a maximum slope of 3H:1V and areas expected to be inundated during the 1% probable annual flood will be lined with bedding and erosion protection materials to mitigate scour. See Section 2.2.2 for further details. Final stabilization measures are required for any exposed Zone I core material, as shown on C1620.

The channel invert begins at the upstream historic cofferdam toe and ends at the known downstream toe of the embankment, as indicated by lidar imagery. If encountered conditions differ from those assumed in this design, the rockfill toe located on the downstream side of the embankment is to be evaluated and graded as required. The concrete cutoff wall will be removed down to bedrock.

2.2.2 EROSION PROTECTION DESIGN

Erosion protection is designed to prevent scour resulting from high-velocity and/or turbulent flow. Erosion protection material is designed in compliance with the USACE (1994) guidelines with a minimum safety factor of 1.2. To prevent erosion of in situ ground located below the slopes lined with erosion protection material, a layer of bedding material will be placed to provide the appropriate filter relationship with the subgrade.

Riprap designs shown on the Drawings were based on 2D hydraulic modeling. Erosion protection for the final river channel at J.C. Boyle is designed for the post-drawdown 1% flood event. Channel characteristics and geometry were used to produce a velocity profile on the left bank and a range of expected velocities from the bottom of the channel to the water surface elevation. It is a requirement that the remaining portion of the embankment be stable, and therefore the fill and cut slopes of the channel must be protected against the long-term design flood. The hydraulics of the final channel were modelled to determine the design parameters for the required slope erosion protection.

The modified Maynord method was used to determine the size and thickness of erosion protection that is required to resist the maximum computed velocity, and the results are verified using other accepted methods. Rock density used for the design of erosion protective layers at J.C. Boyle is assumed to be 165 lb/ft^3. The resulting erosion protection, including D_{50}, layer thickness, key-in and required bedding is shown on Drawing C1230. At the upstream end of the erosion protection reach, the erosion protection will be extended up the bank from a minimum elevation of 3,739.8 ft, where the channel bed begins, to the maximum anticipated river elevation, plus 3 ft of freeboard, of 3,753.0 ft. At the downstream end of the erosion protection reach, the erosion protection will be extended up the bank from a minimum elevation of 3,736.9 ft, where the channel bed begins, to the maximum anticipated river elevation, plus 3 ft of freeboard, of 3,749.0 ft. The expected flood elevations are further discussed in Section 3 of this appendix. The lateral extents of the erosion protection details are shown on Drawing C1230.

The required riprap gradations are presented in the Section 31 05 00 – Materials for Earthwork specification and are also shown on Drawings G0050 and G0051. The United States Department of Agriculture – Part 633 National Engineering Handbook methodology was used to determine bedding size requirements.

2.3 FINAL GRADING

In general, all areas disturbed by construction of the project components will be restored to final lines and grades as soon as practical. All disturbed slopes below the 1% flood elevations will be stabilized with erosion protection or other suitable means.

2.4 DIVERSION CULVERT STOPLOG REMOVAL

2.4.1 BLASTING STOPLOGS

Prior to initiation of Stages 3 and 4 drawdown, charges will be set from the downstream side of the diversion culvert stoplogs. To access the downstream side of the diversion culverts, the spillways must be inactive. To protect the downstream workers and to provide increased reservoir capacity, the spillway gates are to be closed. The timing of the work will depend on inflows into the J.C. Boyle reservoir. The holes will be drilled, and charges set in a manner that conforms with both blasting and demolition technical specifications.

Locations and dimensions of the diversion culvert stoplogs are shown on Drawings C1220 and C1221. The blast design, charging and detonation system will be completed by the Project Company.

2.4.2 DEBRIS MANAGEMENT

Floating debris control measures will be implemented before the drawdown operation commences. PacifiCorp reported that the current debris management measures are successful at reducing the debris load seen at the facility, this includes:

- Debris survey from a boat
- Hand removal of loose woody debris on the shorelines

The water surface levels are expected to stabilize at the staged drawdown surface elevations as defined in Section 3 of this appendix. Debris management and control measures may be implemented at these times to remove any deleterious material that could pose a potential risk to the drawdown and diversion operation.

2.5 CONSTRUCTION ACCESS

2.5.1 GENERAL

Two access roads are designed as part of the 100% DCD: the Powerhouse Road realignment, and the lower penstock access road rehabilitation. The lower penstock access road rehabilitation is an optional temporary upgrade and is defined as road reconstruction. The Powerhouse Road realignment (permanent) are defined as new road construction. The road construction design basis information is included in Design Criteria Appendix A7. It should be noted that an exemption was made to the design criteria grades for portions of the lower penstock access road. The existing maximum road grade is greater in some sections than the maximum design grade stated in the Design Criteria of Appendix A7.

The road construction details are shown on Drawings C1500, C1501, C1511, and C1512.

2.5.1.1 ROAD RECONSTRUCTION

Road reconstruction is the re-opening and upgrade of existing site access roads. Reconstruction upgrades will include brush clearing, ditch cleaning, widening of the road prism, surfacing, and drainage structure installation. Generally, the existing road location will be utilized, minimizing cut-and-fill activities and material quantities. No wearing course or aggregate materials are specified for temporary road reconstruction. The road clearing width will require the removal of vegetation on both sides of the road prism to allow the road surface to dry more readily and to improve visibility on the road. Clearing widths will be minimized, but still allow for safe and stable reconstruction of the road.

2.5.1.2 NEW ROAD CONSTRUCTION

New construction may include falling right-of-way, clearing, and grubbing, stripping, log decking, stump removal, and construction of sub-grade, ditches, and drainage structures. The location of the new roads will be based on topographic features and surface material, and the control point of where that road is accessing. Road grades will be constructed for easier access during construction and to reduce long-term maintenance cost. Similar to reconstruction, new roads will require the establishment of a clearing width or right-of-way.

Subgrade construction consists of using local material to construct a stable surface to form the base of the road. The new roads are assumed to be constructed using a cut-and-fill technique. Material is cut from the uphill side of the road and is placed as fill on the downhill side of the road. The material is then compacted using the tracks of the excavator or bulldozer. New roads are to be capped with Type E11 material with typical sections and details shown on Drawing C6001.

2.5.2 ROAD DRAINAGE

Existing road drainage structures located on project roads (roads to be used for the construction of the project) will be maintained by the Project Company for the duration of the construction period. New drainage structures include drainage swales and drainage culverts as shown on Drawings C1620 to C1624. Swales are required around any design fills to minimize erosion and culverts are required along the power canal alignment to facilitate drainage over the buried concrete. Construction of a matching swale is required on the uphill side of the newly constructed Powerhouse Road Realignment. A culvert exists at the powerhouse which diverts flow from the base of the penstocks and conveys it to the tailrace. If this culvert is removed or blocked during cut-and-fill activities, the Project Company is to replace this culvert with one of equal capacity to convey slope runoff through the powerhouse fill and into the Klamath River. Typical details for bedding and slope of culverts are shown on C1622.

3.0 HYDROTECHNICAL

3.1 RESERVOIR DEPTH-AREA-CAPACITY

The depth-area-capacity relationships for the J.C. Boyle reservoir are based on the 2018 bathymetric survey (NAVD88 datum) and are shown on Drawing C1056. The reservoir capacity at elevations relevant to the J.C. Boyle facility are summarized in Table 3.1.

Table 3.1 Reservoir Storage Capacity for Various Elevations

Key Elevation Description	Elevation(ft)	Capacity(acre-ft)
Maximum Normal Operating Level	3,796.7	3,168
Minimum Normal Operating Level	3,791.7	1,758
Spillway Crest	3,785.2	858
Power Intake Invert	3,771.7	160
Historic Cofferdam Crest	3,770.0	124
Diversion Culvert Invert	3,755.2	0.1

3.2 OUTLET STRUCTURE DISCHARGE RATING CURVES

Discharges during the drawdown stages will be made through the existing outlets at the intake structure: three spillway bays, the power intake, and the two diversion culverts. No alterations will be made to the existing outlets except for the removal of the concrete stoplogs upstream of the two diversion culverts. The development of the discharge rating capacities for the outlets are detailed in Appendix B2. The J.C. Boyle discharge rating curves are presented on Drawing C1056.

3.3 RESERVOIR DRAWDOWN SEQUENCING

The operations of the J.C. Boyle reservoir during drawdown and post-drawdown will achieve successful lowering of the reservoir impoundment and provide the required flood control. The reservoir drawdown sequencing will be completed over four stages as outlined in the design report. The drawdown model (detailed in Appendix G) assesses the drawdown sequencing in terms of reservoir water surface levels under a range of hydrologic conditions. The following sections discuss the results of the drawdown model and the implications to the project.

3.3.1 RESERVOIR CONDITIONS DURING DRAWDOWN AND POST-DRAWDOWN

Reservoir water surface levels are simulated in the drawdown model (Appendix G) for the full record of inflows available for the 2019 Biological Opinion (2019 BiOp) dataset. The 2019 BiOp flows are available for 36 years, from October 1980 through September 2016. The results of the drawdown model are summarized in three ways:

- Individual year simulations are provided in the attached J.C. Boyle Simulated Drawdown Figures 1 through 36. These plots indicate the following:
  - Reservoir water surface levels.
  - Daily average inflows, total outflows, and outflows for each outlet structure (i.e., spillway, power intake, and flows through the diversion culverts).
- Maximum daily reservoir water surface level daily non-exceedance percentiles (percentiles) are shown on Figure 3.1, and on Drawing C1056. This figure represents the results from all 36 model simulations as non-exceedance percentiles to summarize the distribution of the results on any given day of the simulations. These results do not represent a single simulation, but are based on all model simulations.

- Ensemble figures with each line representing a single model simulation for a different year, (also referred to as spaghetti figures) are shown on Figure 3.2. This figure overlaps the simulated reservoir water surface levels on a common x-axis that spans January 1 to September 30. Each line represents a single model simulation.

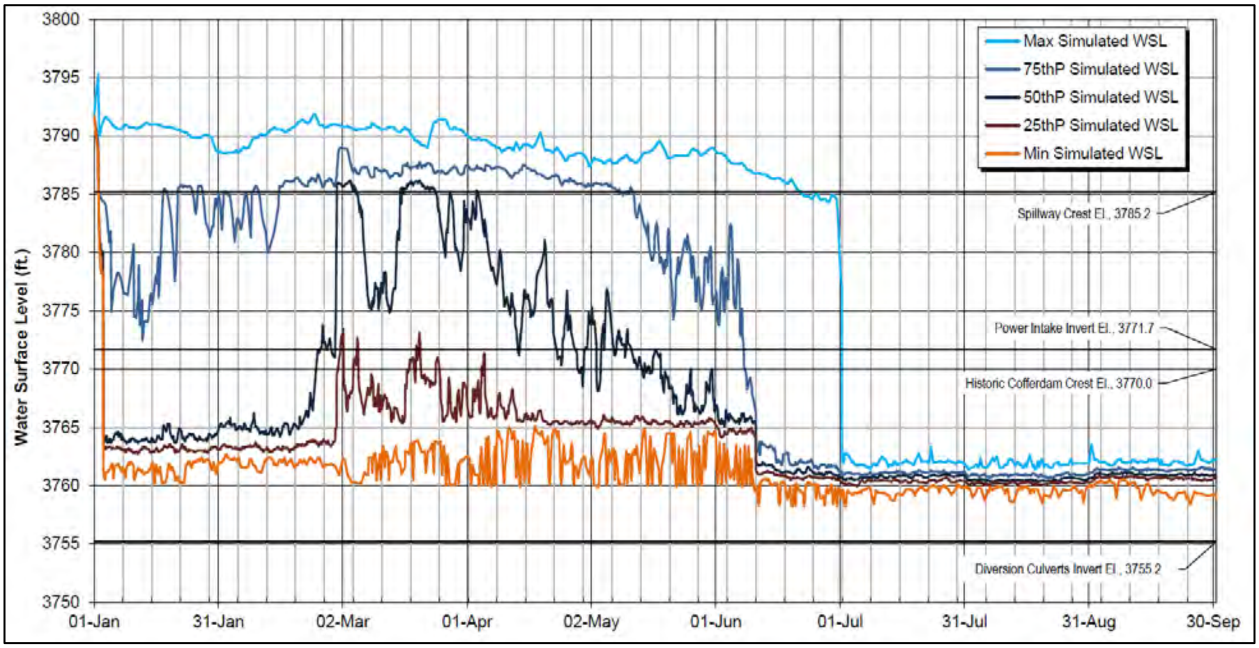


Figure 3.1 J.C. Boyle Reservoir Drawdown Simulated Water Surface Levels Non-Exceedance Percentiles

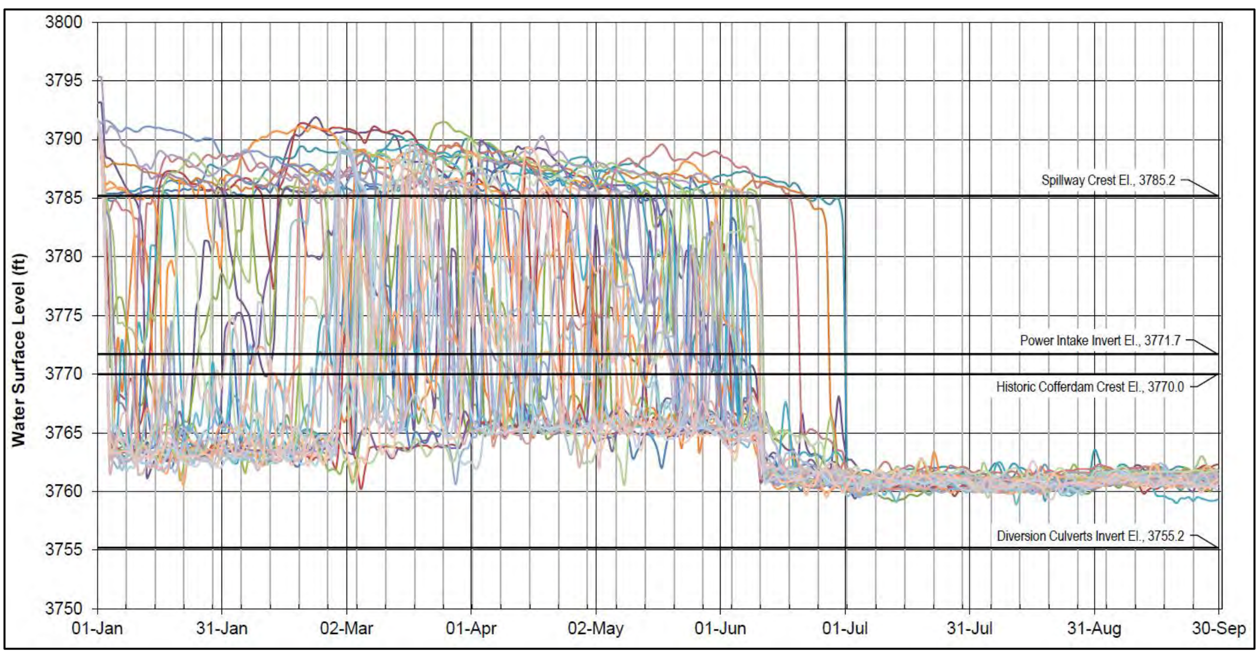


Figure 3.2 J.C. Boyle Reservoir Drawdown Simulated Water Surface Levels Ensemble Plot

The simulated water surface levels on Figure 3.1 and Figure 3.2 show that there is a substantial reduction in the reservoir water levels in mid-June with the majority of the simulated years achieving sustained water levels below the historical cofferdam crest in early June. This is a function of initiating Stage 4 of drawdown on June 10 and the inflow hydrology which indicates a reduction in streamflow for the second half of June (Appendix A6). There are three model years (1983, 1984, and 1998) that show elevated reservoir water surface levels past June 15. However, in these years, the reservoir water surface levels do drop below the crest of the historic cofferdam prior to July 1. Stage 4 can be initiated as early as January once the charges are set on diversion culvert #2 and Stage 3 drawdown is complete, so extending the initiation until June 10 is not a requirement of this design.

Figure 3.2 shows that there are large fluctuations in the reservoir water surface levels from January through June as a function of the inflow hydrology and the J.C. Boyle reservoir. The J.C. Boyle reservoir has a small storage capacity and the reservoir can refill quickly during the higher flow months typically in January through May resulting in spillway flows. Lower reservoir levels will be sustained below the crest of the historic cofferdam during Stage 4 and after June 1 depending on the hydrologic conditions.

Figure 3.3 shows reservoir drawdown distributions for various relevant facility components, which represent cumulative percentages of model simulations indicating the dates when the reservoir water surface level is lower and sustained below a certain elevation. The actual date when the water surface elevation will be sustained in the drawdown year can be different than shown on Figure 3.3, depending on the hydrological conditions and the drawdown sequencing applied. The water levels shown on Figure 3.3 are based on average daily conditions for the 36 drawdown model simulations. Low probability flood flows (e.g., the 5% or 1% probable flood flows) may have not occurred within this period and may not be reflected in these drawdown distributions. Occurrence of such events may shift the distributions to a later date. The following observations are made based on Figure 3.3:

- Elevation 3,792.1 ft – represents embankment phase 2 crest, at which point the embankment removal down to the June 1 1% probable flood at elevation 3,790.0 ft can start. Approximately 97% of the simulations have reservoir water levels sustained below the embankment Phase 2 crest by January 2, and 100% of the simulations by January 3.
- Elevation 3,785.2 ft - represents the spillway crest. Approximately 45% of the drawdown simulations have reservoir water levels sustained below the spillway crest by April 20, 91% of the simulations by June 10, and 100% of the simulations by June 20. This indicates that diversion culverts become more accessible during the late spring and summer months.
- Elevation 3,771.7 ft – represents the power intake invert. Approximately 40% of the simulations have reservoir water levels sustained below the power intake invert by June 1, 91% of the simulations by June 10, and 100% of the simulations by July 1.
- Elevation 3,770.0 ft – represents the crest of the historic cofferdam. Approximately 40% of the simulations have reservoir water levels sustained below the crest of the historic cofferdam by June 1, 91% of the simulations by June 10, and 100% of the simulations by July 1. This indicates that the height of the assumed cofferdam is appropriate for diverting flows during the anticipated embankment removal construction window.

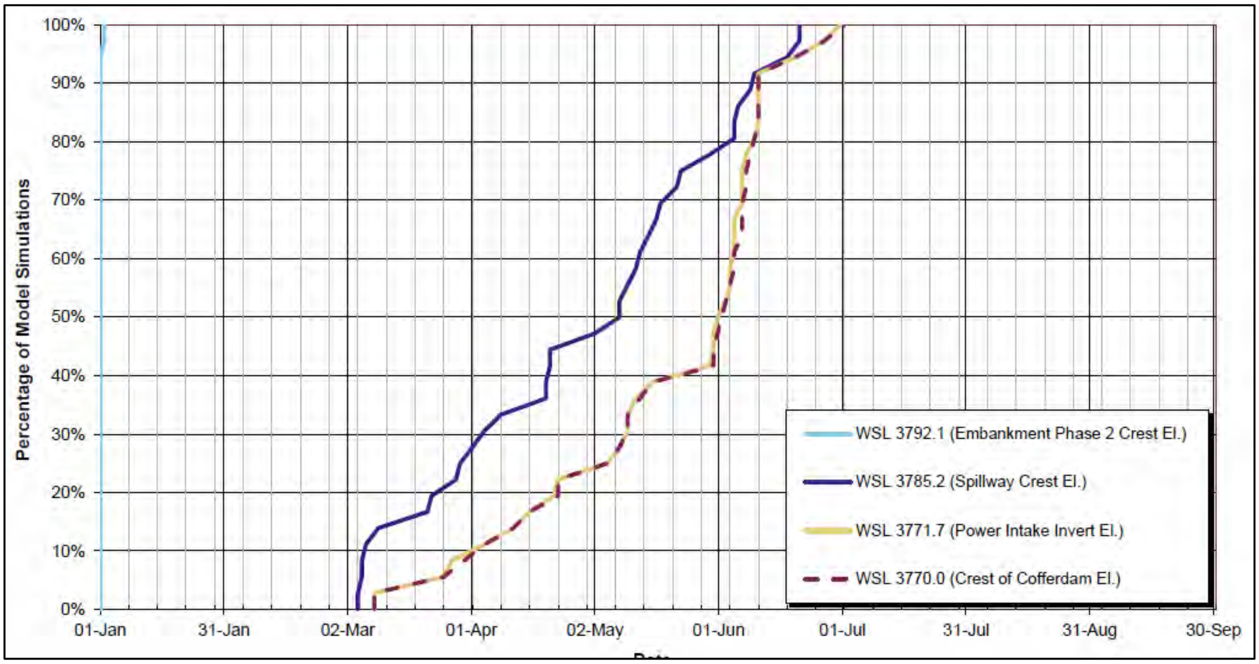


Figure 3.3 J.C. Boyle Reservoir Drawdown Cumulative Model Simulation Dates to Achieve and Sustain Reservoir Water Surface Levels below Various Relevant Elevations

The results of the reservoir drawdown model are outlined below for each stage of drawdown. It should be noted that the rules set for the drawdown model do not include coordination with the Upper Klamath River Basin (Keno Dam and/or Klamath Lake) or initiation of Stage 4 drawdown prior to June 10 which is acceptable if Stage 3 drawdown is complete and the charges have been set on the downstream side of the second diversion culvert.

- Stage 1 - Spillway Gates and Power Intake:
    - The spillway gates and power intake are used to target a drawdown of 5 ft/day, and drawdown occurs over one to two days to reach the spillway crest (El. 3,785.2 ft).
- Stage 2 - Power Intake:
    - The reservoir water levels are controlled by the discharge capacity of the power intake and are dependent on the reservoir inflows.
    - Outflows through the power intake are limited to 2,850 cfs. The total outflow can be higher if the spillway is still engaged.
    - The reservoir can be lowered up to 5 ft when the power intake is initially opened in drier climatic conditions, as seen in the simulated results for 1990 and 2015.
    - The drop in reservoir water surface levels is not as large in wetter climatic conditions, and the water level may be maintained above the spillway crest, as seen in simulated results for 1984 and 1997.

- The duration of Stage 2 is determined by the hydrologic conditions and when the downstream side of the diversion culverts can be accessed to successfully remove the stoplogs. Approximately 75% of the simulations indicate that the duration of Stage 2 is limited to less than a week under the simulated drawdown methodology. In approximately 10% of simulations, Stage 2 was limited to 2 weeks (1982, 1996, 1998, and 2002). Years with much higher than average inflows (wet years) indicate that Stage 2 can be extended for many weeks and beyond April 1 into May and June. This is observed in less than 15% of the simulated years (1983, 1984, 1985, 1997, and 2006). In case such a wet year occurs during the drawdown year, the demolition of the powerhouse and conveyance facilities at J.C. Boyle could be delayed, unless the inflows to the reservoir are managed by coordinating with the Upper Klamath River Basin (Keno Dam and/or Klamath Lake).
- River forecasting and coordination with the Upper Klamath River Basin may be required to limit the duration of Stage 2. Reduced inflows to the reservoir will result in lower reservoir water levels, therefore, allowing for safe access to the downstream end of the diversion culverts. The steady-state inflow to the reservoir to maintain a water level at the spillway crest with the power intake is approximately 1,600 cfs for Stage 2. Alterations to the flow releases from the Upper Klamath River Basin outside of the 2019 BiOp flows were not simulated with the drawdown model.

- Stage 3 – Opening of the First Diversion Culvert:
  - A temporary drop in reservoir water surface level and an increase in outflow is observed when the diversion culvert is opened. The reservoir water surface levels will drop below 3,765 ft under most hydrological conditions when the diversion culvert is opened. Wetter hydrological conditions will result in a lesser drop in the reservoir level (e.g., 1998 drops to approximately 3,770 ft as there is an increase in reservoir inflows shortly after removing the diversion culvert stoplogs).
  - Outflows through the diversion culvert are limited to approximately 2,400 cfs prior to engaging the spillway crest. Total outflows in Stage 3 can be higher if the spillway is still engaged.
  - The reservoir water surface level is likely to increase periodically after opening the first diversion culvert. Nearly 90% of the model simulations indicate that the spillway will be reengaged at some point during Stage 3 if the initiation of Stage 4 is extended until June 10.
  - The drawdown model report (Appendix G) notes that under the drawdown operating criteria evaluated for the drawdown model, in some years both diversion culverts open on the same date (June 11). Under these hydrological conditions, coordination with the Upper Klamath River Basin would be required to permit the opening of the first diversion culvert on an earlier date, therefore initiating Stage 3 of drawdown prior to June 10.
- Stage 4 – Opening of the Second Diversion Culvert:
  - Stage 4 represents the final stage of drawdown.
  - Stage 4 is initiated on or after June 10 and when the reservoir water surface level below the spillway crest. The steady-state inflow to the reservoir to maintain a water level at the spillway crest with the first diversion culvert open is 2,210 cfs.
  - Over 90% of the drawdown model simulations indicate that the second diversion culvert is opened on June 10. Under wet hydrological conditions, such as those in simulation years 1983, 1984, and 1998, the opening on the diversion culvert is delayed – the latest date resulting from the simulations is June 29.

- The reservoir water surface levels will drop below 3,763 ft under most hydrological conditions when the second diversion culvert is opened. Wetter hydrological conditions will result in a lesser drop in the reservoir level (e.g., 1993, 1998, 1999 and 2011 drops to approximately 3,765 ft with the initial opening of the diversion culvert).
- After the diversion culvert has been opened, and after July 1, the reservoir water surface levels remain low and are within the range of 3,758 ft to 3,763.5 ft for all model simulations.

3.3.2 SCOUR POTENTIAL

The CFD study presented in Appendix B2 indicates that during the drawdown operation there will be varying levels of scour potential immediately upstream of the outlet structures. Scour potential was predicted by comparing simulations of bed shear stress from both the Flow-3D and HEC-RAS 2D models. Shear stress magnitude figures are shown in Appendix B2.

It is anticipated that the flow will have the potential to scour medium to very coarse gravels (up to 1.3 inches) during Stage 2 and Stage 3 of drawdown. The bed shear stresses in the reservoir stimulated during Stage 4 have the potential to mobilize small cobbles (2.5 inches). The scour potential is the highest at low flows when a minimal headpond is present, resulting in the ability to mobilize large cobbles (5-10 inches). Given the 9.5 ft by 10 ft opening of the diversion culverts, blockage from mobilized bed material is not anticipated.

Modelled flow paths are shown on Figure 12 and Figure 13 in Appendix B2 during low flows. It should be noted that due to current sediment deposits, water is flowing directly over the historic cofferdam rather than being diverted around it. It is anticipated that scour of the bed will alter the current sediment geometry. Mechanical evacuation of sediment may be necessary and is to be evaluated post-drawdown.

3.4 EMBANKMENT REMOVAL AND STEADY-STATE WATER SURFACE LEVELS

3.4.1 GENERAL

Design criteria and hydrology determine the projected water surface levels at the J.C. Boyle Facility. The water surface levels ultimately dictate the removal schedule for the components of the facility in direct contact with the Klamath River. The only components of the facility in direct contact with the river are at the J.C. Boyle dam – namely the intake, spillway, diversion culverts, embankment, and historic cofferdam. The steady-state water surface levels at these components are presented below.

3.4.2 INTAKE AND EMBANKMENT WATER SURFACE ELEVATIONS

Flood water surface levels at the intake and embankment are shown on Drawing C1055 for steady-state inflows. The statistical flood flows (high water) are based on peak instantaneous flows while the daily average flows are average flows over a 24-hour period. The levels are calculated using the discharge rating curves developed for the outlet structures (Drawing C1056 and Appendix B2). The levels may differ depending on the shape and volume of the flood flow hydrographs and the attenuation effects of the reservoir. It should be noted that the spillways must remain operational until June 15, during which time the 1% probable flood water surface level with freeboard is at or above the spillway invert elevation of 3,785.2 ft.

The water levels at the intake are lower compared to the embankment water levels for flows less than 3,000 cfs, and at low flows, the water surface levels at the intake and embankment will be visually distinguishable, as discussed in Section 3.3.2 above and Appendix B2.

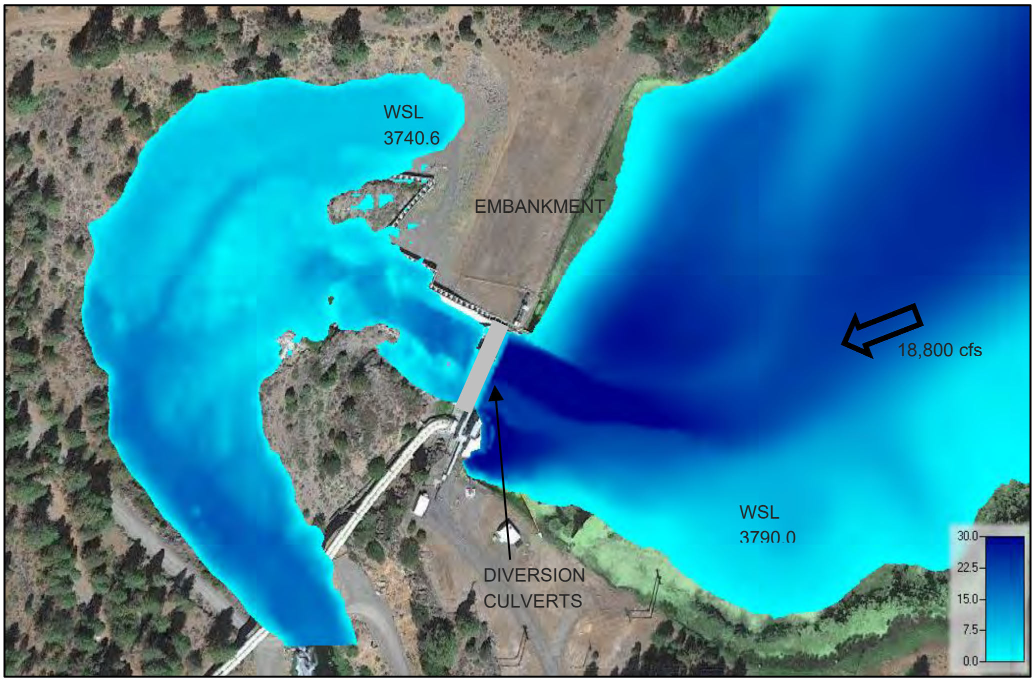
3.4.3 EMBANKMENT AND HISTORIC COFFERDAM WATER SURFACE ELEVATIONS

Flood water surface levels at the embankment and historic cofferdam are shown on Drawing C1231. The levels shown correspond to the embankment steady-state water levels as shown on C1055. The Staged crest elevations have been designed for the 1% flood with 3 or 1 ft freeboard (depending on the period). This assumes that the sediment, in its current state, remains and is not removed via scour during drawdown.

Little is known on the design or condition of the historic cofferdam. A 2018 bathymetric survey (NAVD88 datum) indicates that areas of the dam are partially eroded. Some fill placement may be required to provide a uniform crest width and slopes following cofferdam assessment post-drawdown. The staging design assumes that the crest elevation is 3,770.0 ft which is anticipated to contain all 1% flow events between July 16 and September 15 and all 5% flow events between June 15 and September 30. Mechanical evacuation of sediment may be necessary to ensure the functionality of the cofferdam.

3.4.4 EMBANKMENT TAILWATER LEVELS

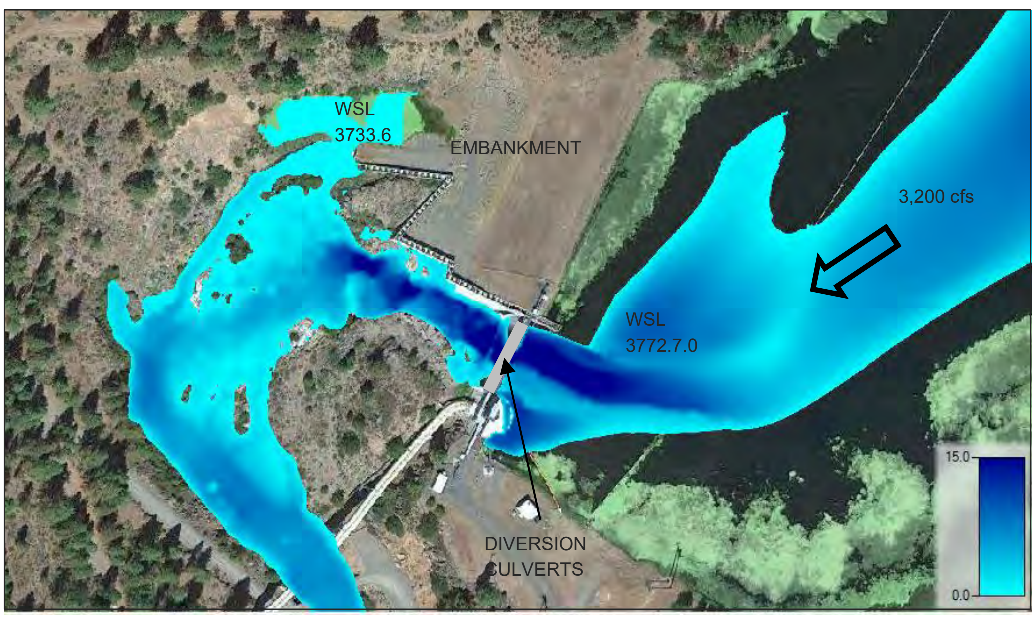
A hydrodynamic model was developed to investigate the tailwater surface levels downstream of the embankment once both diversion culverts are opened post-drawdown. The model was completed using HEC-RAS 2D with a Manning's n of 0.04. The model was run at a flood of 18,800 cfs, which is greater than the 1% probable annual flood, to evaluate the potential of backwatering of the downstream toe of the embankment. The model was also run at a flood of 3,200 cfs, which is the largest monthly 1% probable flood from July 1 to September 30, to evaluate the water surface levels on the downstream side of the embankment during the deconstruction period. The resulting tailwater levels indicate that the pond at the downstream toe of the dam could be hydraulically connected to the diversion culvert outflow via the original river channel during large storm events. The resulting water depths and water surface levels are shown on Figure 3.4 and Figure 3.5 for the 18,800 cfs and 3,200 cfs floods, respectively.



NOTES:

1. DEPTHS ARE IN FEET.
2. WSL: WATER SURFACE LEVEL.

Figure 3.4 J.C. Boyle Reservoir Tailwater Depths – Drawdown Stage 4



NOTES:

1. DEPTHS ARE IN FEET.
2. WSL: WATER SURFACE LEVEL.

Figure 3.5 J.C. Boyle Reservoir Tailwater Depths – Deconstruction Period

3.4.5 KEY INTAKE AND EMBANKMENT REMOVAL TIMING

Calculated water surface elevations and design criteria are used to determine the earliest removal date for key intake and embankment removal items in Table 3.2. The embankment removal work is broken up into phases that represent dates corresponding to water surface levels. Each phase has a designated removal volume and the staged elevations are shown on Drawings C1231 and C1234 to C1239.

Table 3.2 Key Intake and Embankment Elevations and Removal Timing

Removal Item	Elevation (ft)	Design Flood Event	Earliest Removal Date	Phased Volume (yd^3)	Haul Location	Comments
Spillway Gates and Trunnions	3,790.0	-	Jan 1	-	Scour Hole	Trunnions and spillway gates are not necessary for spillway operation and can be removed after drawdown.
Diversion Culvert #1 (Drawdown Stage 3)	3,755.2	-	Varies	-	-	See drawdown section (Stage 3).
Embankment Removal **Phase 1**	-	1% Probable Flood + 3 ft freeboard	Mar 15	4,400	Penstock Cover and Scour Hole	Remove erosion protection material from downstream face of the dam.
Embankment Removal **Phase 2**	3,792.1	1% Probable Flood + 3 ft freeboard	Jun 1	8,500	Penstock Cover and Scour Hole	Remove embankment to June 1 1% probable flood with 3 ft freeboard
Diversion Culvert #2 (Drawdown Stage 4)	3,755.2	-	Varies	-	-	See drawdown section (Stage 4).
Embankment Removal **Phase 3**	3,784.7	1% Probable Flood + 3 ft freeboard	Jun 15	13,200	Penstock Cover and Scour Hole	Remove embankment to June 15 1% probable flood with 3 ft freeboard
Spillway Structure	3,785.2	1% Probable Flood + 3 ft freeboard	Jul 1	-	Scour Hole	Remove spillway and intake structure to max removal elevation – maintain 15 ft width for access to left bank
Abutment Left Wall Phase 1	3,785.2	1% Probable Flood + 3 ft freeboard	Jul 1	-	Scour Hole	Match left wall elevation to spillway and elevation.
Embankment Removal **Phase 4**	3,776.7	1% Probable Flood + 3 ft freeboard	Jul 1	18,700	Left Bank Disposal Area	Remove embankment to July 1 1% probable flood with 3 ft freeboard
Embankment Removal **Phase 5**	3,770.7	1% Probable Flood + 1 ft freeboard	Jul 15	17,930	Left Bank Disposal Area	Criteria changes from 1% probable flood with 3 ft freeboard to 1% probable flood with 1 ft freeboard. Remove embankment to July 15 1% probable flood with 1 ft freeboard
Embankment Removal **Phase 6**	3,770.0	1% Probable Flood + 1 ft freeboard	Aug 1	120,832	Left Bank Disposal Area	Remove remaining embankment and silt. Excavate final channel to lines and grades shown on C1230. Stockpile material for eventual placement in diversion culvert channel and to bury intake concrete (Phase 9).
Evaluate/Grade Downstream Rockfill **Phase 7**	3,770.0	1% Probable Flood + 1 ft freeboard	Aug 1	-	-	Evaluate rockfill for use in final channel following removal Phase 6 and grade as required.
Install Erosion Protection **Phase 8**	3,770.0	1% Probable Flood + 1 ft freeboard	Aug 1	-	-	Install erosion protection and bedding material on the left bank of the final river channel.
Historic Cofferdam Breach **Phase 9**	3,755.2 (min)	-	Sep 1	4,900	Right Bank Disposal Area	To start no earlier than September 1 and be completed no later than September 30. Breaching of the historic cofferdam must take place after the final channel excavation and erosion protection installation is substantially complete.
Intake Cover **Phase 10**	-	-	Sep 1	5,000	Left Bank Disposal Area	To occur after cofferdam breach and substantial completion of the Final River Channel. Place material in diversion culvert channel and bury intake concrete.

3.5 FINAL RIVER CHANNEL WATER LEVELS

Channel characteristics and geometry of the J.C. Boyle final river channel presented in Section 2.2 and on Drawings C1230 and C1232 were used to develop a hydrodynamic model to determine the stage-discharge relationship for the post-dam removal period.

The resulting stage-discharge relationship is shown on Figure 3.6 at the location of the current dam centerline. A sensitivity of the model was completed using Manning's n roughness of 0.03 and 0.06 to account for potential variability in roughness elements added to the channel to provide localized habitat elements. The results of the sensitivity analysis are also included on Figure 3.6.

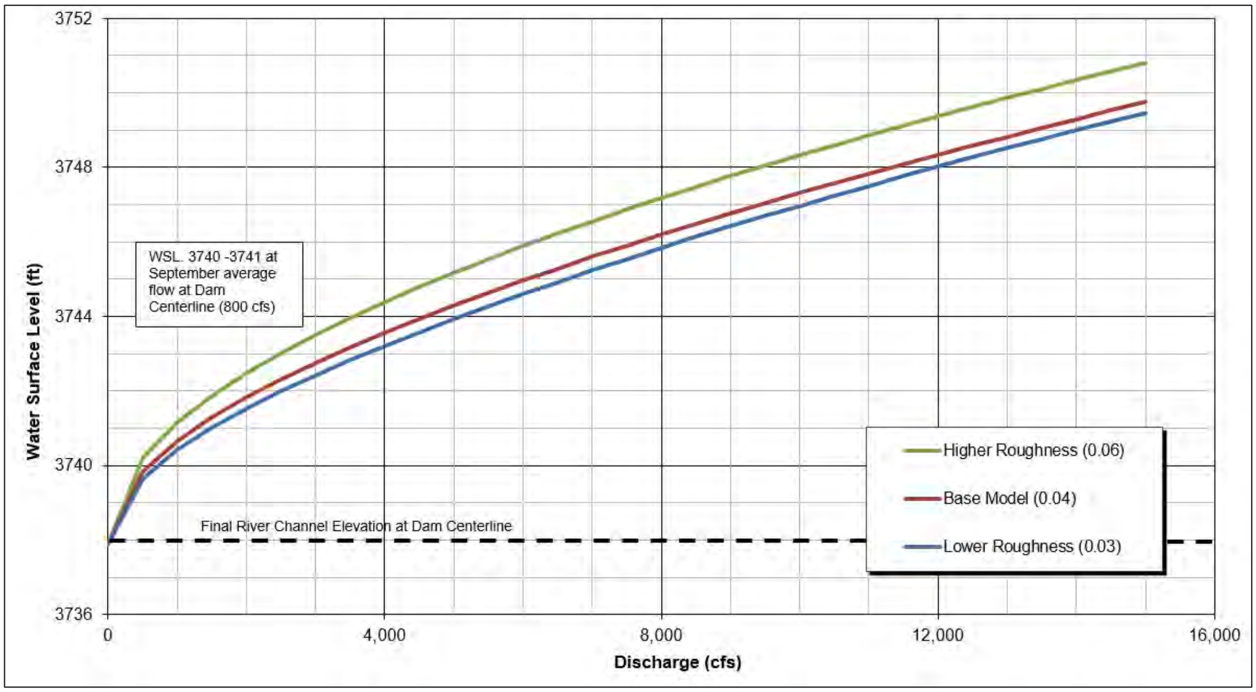


Figure 3.6 Final River Channel Stage-Discharge Relationship at Existing Dam Centerline

Dam removal construction activities in the vicinity of the final river channel are scheduled to continue into the fall. Steady-state water surface levels for probable floods and mean monthly flows for specified periods in September through November, are provided for reference in Table 3.3 using the base model Manning's n value of 0.04.

In addition, steady-state water surface levels for the final river channel for the annual probable floods, the mean annual flow, and the annual 25% and 75% flow durations are provided in Table 3.4 using the base model Manning's n value of 0.04.

Table 3.3 Final River Channel Monthly Steady-State Water Surface Levels at Existing Dam Centerline

Flow Condition		Discharge (cfs)					
		Time Period					
		Sep 1 - 15	Sep 16 - 30	Oct 1 - 15	Oct 16 - 31	Nov 1 - 15	Nov 15 - 30
Statistical High Water (Flood Conditions)	5% Probable Flood	2,100	2,400	2,900	3,300	3,800	4,400
	20% Probable Flood	1,700	1,900	2,200	2,400	2,600	2,900
	50% Probable Flood	1,400	1,500	1,700	1,700	1,800	2,000
Mean Flow for Specified Time Period		800	790	810	890	980	950
Flow Condition		Water Surface Levels (ft) - Post-Dam Removal at Dam Centerline[1]					
Statistical High Water (Flood Conditions)	5% Probable Flood	3,741.9	3,742.2	3,742.7	3,743.0	3,743.4	3,743.9
	20% Probable Flood	3,741.5	3,741.7	3,742.0	3,742.2	3,742.4	3,742.7
	50% Probable Flood	3,741.2	3,741.3	3,741.5	3,741.5	3,741.6	3,741.8
Mean Flow for Specified Time Period		3,740.3	3,740.3	3,740.3	3,740.5	3,740.6	3,740.6

NOTES:
1. FINAL RIVER CHANNEL BED AT DAM CENTERLINE IS AT ELEVATION 3,738 ft.

Table 3.4 Final River Channel Annual Steady-State Water Surface Levels at Existing Dam Centerline

Flow Condition		Discharge (cfs)
Statistical High Water (Flood Conditions)	1% Probable Flood	14,200
	5% Probable Flood	11,700
	20% Probable Flood	8,500
	50% Probable Flood	7,000
Annual Flow Duration 25% of Time Equaled or Exceeded		1,460
Mean Annual Flow		1,390
Annual Flow Duration 75% of Time Equaled or Exceeded		690
Flow Condition		Water Surface Levels (ft) - Post-Dam Removal at Dam Centerline
Statistical High Water (Flood Conditions)	1% Probable Flood	3,749.4
	5% Probable Flood	3,748.2
	20% Probable Flood	3,746.5
	50% Probable Flood	3,745.6
Annual Flow Duration 25% of Time Equaled or Exceeded		3,741.2
Mean Annual Flow		3,741.1
Annual Flow Duration 75% of Time Equaled or Exceeded		3,740.1

NOTES:
1. FINAL RIVER CHANNEL BED AT DAM CENTERLINE IS AT ELEVATION 3,738 ft.

3.6 TAILRACE BACKFILL

3.6.1 STEADY-STATE WATER SURFACE LEVELS

A hydrodynamic model was developed to investigate the water surface levels at the backfilled tailrace once both diversion culverts are opened post-drawdown. The model was completed using HEC-RAS 2D with a Manning's n of 0.04. The model was run with average June and July flows, which corresponds to the anticipated period of construction, to evaluate the water surface levels on the tailrace buttress during the construction period. The resulting water surface levels indicate that the average water surface level for June and July has an approximate elevation of 3331.0 ft. This elevation is shown on Drawing C1411.

3.6.2 EROSION PROTECTION DESIGN

Using the modified Maynord method as described in Section 2.2.2, the tailrace erosion protection is designed for the post-drawdown 1% probable annual flood event. Channel characteristics and geometry was used to produce a velocity profile on the tailrace buttress and a range of expected velocities. It is a requirement that the infilled tailrace outer slope be stable, and therefore the fill slope must be protected against the long-term design flood. The 1% flood event water surface level is shown on C1411 and the analysis indicates that the outer 2 feet of the fill slope must consist of a low fines material. The maximum velocity is estimated to be 1 ft/s during the 1% flood event, assuming a Manning's n value of 0.04.

4.0 GEOTECHNICAL

4.1 DAM EMBANKMENT STABILITY DURING DRAWDOWN

Stability of the dam embankment during reservoir drawdown was assessed by Limit Equilibrium Analysis (LEA) for transient pore pressure distributions produced by a generalized drawdown curve, which was defined based on results from the drawdown simulations (1981 through 2016). GeoStudio (GEO-SLOPE, 2020) was used to complete the seepage analysis and LEA (Spencer and Morgenstern-Price or GLE method of slices). The acceptance criterion is defined by a Factor of Safety (FOS) of 1.3, based on the more conservative recommendation of the USACE (2003).

The drawdown simulations indicated variable drawdown rates and multiple drawdown-refill cycles that could involve sizeable changes in water level elevation over a short duration. As a result, a generalized curve was defined to drawdown at the fastest simulated rate for the largest total head difference and to provide sufficient time for re-saturation on reservoir refilling. The full curve is shown in the inset of Figure 4.1 and the stability analyses, at 1-day timesteps for the corresponding transient seepage analyses, indicated the lowest FOS occurred at the initial reservoir drawdown. Consequently, a higher resolution drawdown curve was developed for the first eight days of the full drawdown curve with 1-hour timesteps for the transient seepage analyses. The refined curve is shown on Figure 4.1.

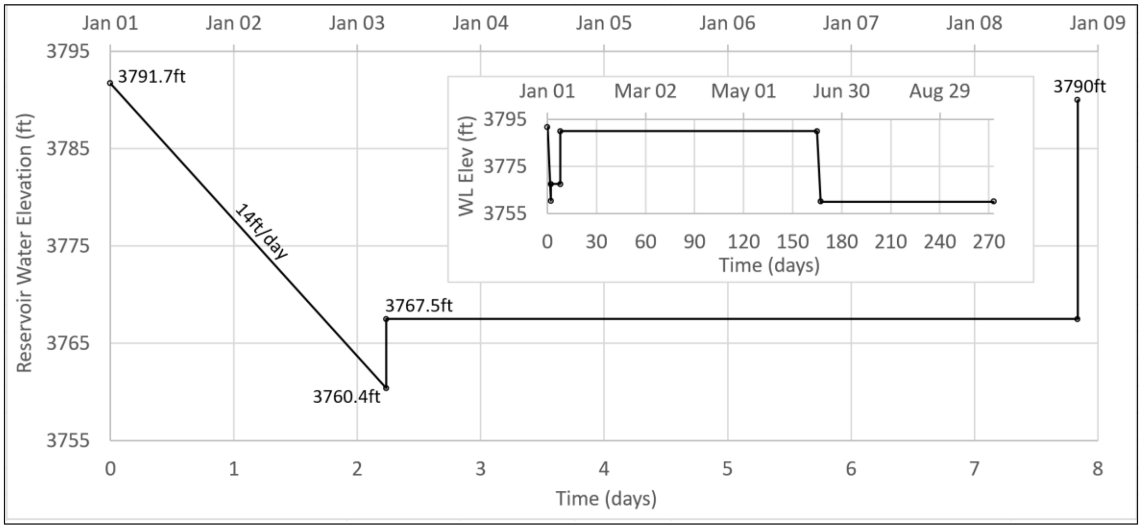


Figure 4.1 Generalized Drawdown Curve For Stability Analyses

Material properties used in the analyses are shown in Table 4.1 and include both base case and sensitivity values. A sensitivity check was completed on a second model by making changes to material properties that are shown in Table 4.1.

Table 4.1 Material Properties for Drawdown Stability Assessment

Material	Horizontal Hydraulic Conductivity (ft/s)	Vertical:Horizontal Hydraulic Conductivity Ratio	Unit Weight (pcf)	Effective Friction Angle (°)	Effective Cohesion (psf)
Core	5.61e-06	1 (0.1)	120	27 (19)	0
Shell	2.17e-02 (1.32e-04)	1 (0.5)	130	34	0
Upstream Riprap	3.41e-02 (1.32e-04)	1	140	34	0
Downstream Riprap	3.41e-02	1	140	34	0
Filter Blanket	3.41e-02	1	125	35	0
Waste Rock Fill	2.17e-02	1	145	40	0
Bedrock	3.28e-10	1	impenetrable		

NOTES:

1. SENSITIVITY CHECKS COMPLETED WITH VALUES IN PARENTHESES.

The analysis model geometry is shown on Figure 4.2. Three scales of slip were considered in the LEA. The first was a full-height slip, extending from the dam crest to the upstream toe. The second was a smaller slip, involving the lower slope or from the bench at elevation 3783.7 ft to the upstream toe. The third was a smaller slip that involved the upper slope, extending from the dam crest to the bench at elevation 3783.7 ft.

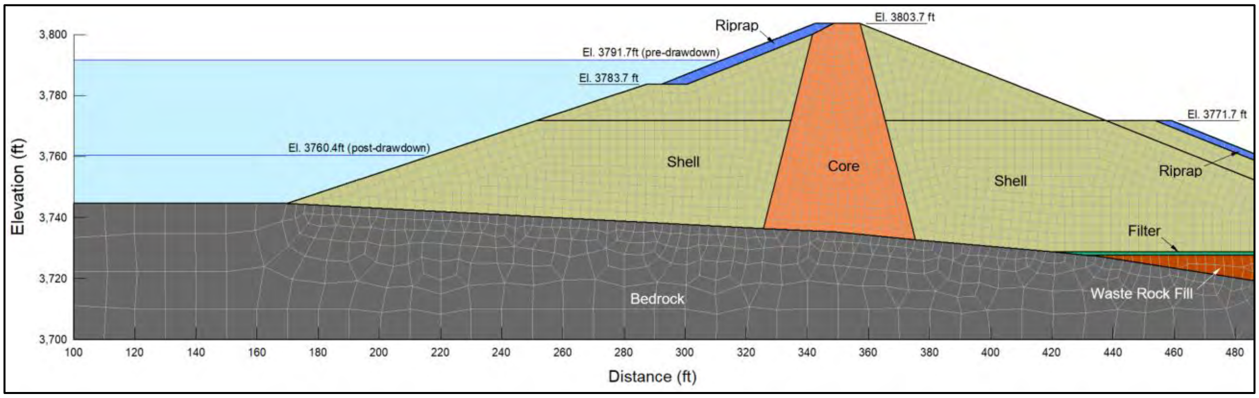
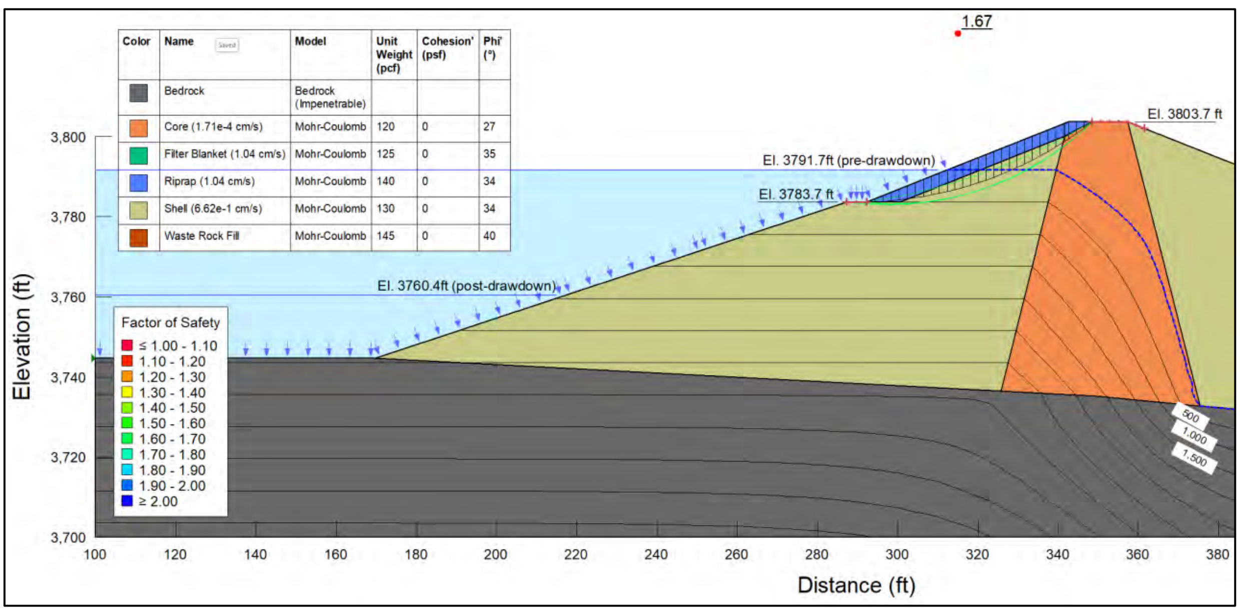


Figure 4.2 Drawdown Analysis Model Geometry

The stability results indicate the lowest FOS for the three scales of slips is 1.7 for the base case properties and 1.4 for the sensitivity check. Both values correspond to the GLE method, which produced slightly lower FOS values than the Spencer method. These results indicate the dam embankment is expected to be stable during the defined drawdown curve.

The upper slope slip governs stability for base case properties with the critical slip shown on Figure 4.3, along with pore pressure contours in psf. The timestep associated with the critical slip is at the beginning of drawdown.



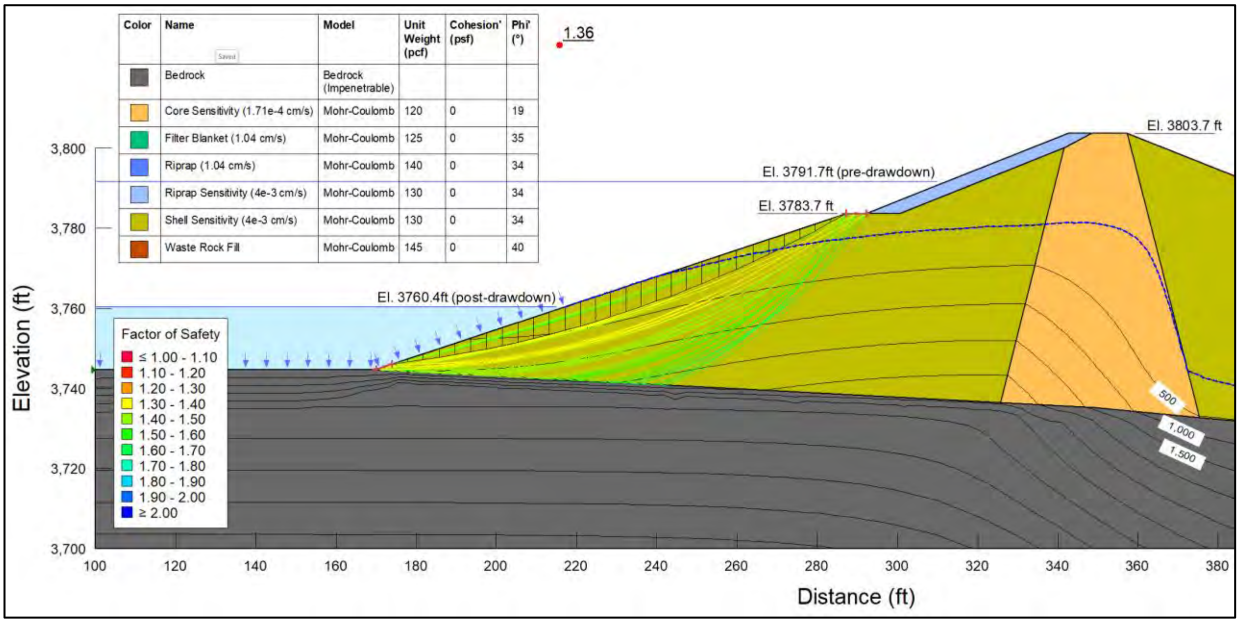
Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Bedrock	Bedrock (Impenetrable)			
	Core (1.71e-4 cm/s)	Mohr-Coulomb	120	0	27
	Filter Blanket (1.04 cm/s)	Mohr-Coulomb	125	0	35
	Riprap (1.04 cm/s)	Mohr-Coulomb	140	0	34
	Shell (6.62e-1 cm/s)	Mohr-Coulomb	130	0	34
	Waste Rock Fill	Mohr-Coulomb	145	0	40

NOTES:

1. ONLY SLIPS WITH FOS < 1.7 ARE SHOWN.

Figure 4.3 Drawdown Stability Results for Upper Slope Slip with Base Case Properties

The lower slope slip governs stability for the sensitivity check. The critical slip is shown on Figure 4.4 with pore pressure (psf) contours included. The timestep associated with the critical slip is about 2.2 days after drawdown commences, which coincides with the minimum water elevation (3,760.4 ft) used in the analyses.



NOTES:

1. ONLY SLIPS WITH FOS < 1.5 ARE SHOWN.

Figure 4.4 Sensitivity Check of Drawdown Stability Results for Lower Slope Slip

Stability results are summarised in Table 4.2 for the three scales of slips. Results for the GLE method are reported since it produced the lower FOS values.

Table 4.2 Factor of Safety Results for Drawdown Stability

Slip Scale	Base Case Properties	Sensitivity Check
Full Height	1.85	1.49
Lower Slope	1.87	1.36
Upper Slope	1.67	1.58

4.2 EXCAVATION SLOPES

4.2.1 STABILITY OF THE EMBANKMENT DURING EXCAVATION

The embankment will be removed from the top down, maintaining the current slopes and toes of the embankment. Following excavation of the embankment to elevation 3,770.7 ft, the final portion of the embankment will be excavated as shown on Drawing C1238 to create the final river channel. This excavation methodology is prescribed to ensure that embankment stability is maintained through the removal process, until the embankment is substantially removed. The embankment removal sequence is shown on Drawings C1231 and C1234 to C1239.

4.2.2 RIVER CHANNEL

The left bank of the channel is excavated in embankment fill materials, while the right bank and channel bottom are excavated to bedrock. This appendix describes the geotechnical considerations of the design of the left bank excavated slopes in the embankment cut. Refer to Section 2 of this appendix for the civil design of the river channel.

The existing embankment is comprised of an exterior layer of riprap and bedding, Zone II shell zones and a Zone I core zone. The riprap and bedding materials were investigated, as summarised in the Geotechnical Data Report (VA103-640/1-2) and is expected to meet E7 erosion protection and E8 bedding material specifications but will require sorting by particle size. Zone II is expected to meet the criteria of General Fill – Type E9b, while Zone I is expected to meet the criteria of General Fill – Type E9 with no processing. Material specifications are available in the Project Technical Specifications and shown on Drawings G0050 and G0051. The following excavation slopes criteria are used in the design:

- Min. 1.5 H:1.0 V excavation slopes for all slopes located in alluvium and highly weathered rock.
- Min. 1.0 H:1.0 V excavation slopes for all slopes located in weathered rock.
- Min. 0.5 H: 1.0 V excavation slopes for all slopes located in competent rock.

Riprap and bedding materials stored from embankment excavation may be used as erosion protection for the final river channel. Dam core Zone I left in place will require final stabilization treatment as shown on Drawing C1620.

Stability of the final excavated slope of the dam embankment at the left bank was assessed by LEA for two loading conditions: static long-term and yield acceleration (ky) determination for approximating seismic displacement. The acceptance criteria require a FOS of 1.5 for static long-term stability and FOS of 1.0 for yield acceleration determination without strength reduction. In addition, STID-8 (PacifiCorp, 2015a) indicated displacements of 2 ft are acceptable according to a FERC guideline for the operating dam.

Static short-term stability was not analyzed since the design deconstruction staging entails embankment removal in horizontal layers from the right bank towards the left bank. This sequence and specific direction, in combination with expected pore pressure dissipation following reservoir drawdown, should provide reasonable confinement (buttressing) while promoting pore pressure dissipation due to controlled exposure of the core material. Pseudo-static analyses were precluded since seismic displacements were approximated from the yield acceleration determined from the LEA geometry.

The LEA was completed in three dimensions with Slide3 (Rocscience, 2020) and using both Spencer and GLE methods. A dry slope was assumed for the piezometric conditions. The design seismic loading was defined by STID-5 (PacifiCorp, 2015b), for a Maximum Credible Earthquake (MCE) of 0.55 g, and STID-8 (PacifiCorp, 2015a), for a magnitude 7.25 earthquake. Seismic displacements were approximated by two semi-empirical methods, developed by Makdisi and Seed (1978) and Bray and Travasarou (2007).

Material properties used in the analyses are summarised in Table 4.3. The dam embankment materials (core and shell) were adopted from STID-8 (PacifiCorp, 2015b) except for the shear strength of the shell. Previous analyses completed for the Definite Plan (KRRC, 2018) considered a reduced strength (from 37°) based on an evaluation of construction records. The reduced value was adopted for the design analysis.

Table 4.3 Material Properties for Left Bank Final Dam Excavation Stability Analysis

Material	Unit Weight (pcf)	Effective Friction Angle (°)	Effective Cohesion (psf)
Core	120	27	0
Shell	130	34	0
E4 Cap	125	35	0
E7 Erosion Protection	90	Leps (1970) Lower-bound Shear-Normal Function	

The model was simplified to assume the foundation units comprised only the dam core and shell materials. This simplification did not affect the results as the critical slips were located wholly within the remaining dam embankment.

The GLE method provided more conservative results and was used for assessing the design of the permanent slope. The yield acceleration slip search considered two scales. The smaller-scale shallow slip resulted in the lowest ky (0.17) and extends approximately two-thirds of the embankment height. The larger-scale slip (full height) resulted in a larger ky of 0.22. Displacement estimates indicate the shallow slip will likely displace greater than 2 ft. However, the predicted slip is small in volume (500 yd^3) and shallow in depth (5 ft at its maximum). The consequence of this size of failure is not expected to dam the river. The larger-scale slip, approximately 15 ft deep with a volume of 4,200 yd^3, is predicted to displace less than 2 ft.

The static FOS for a full-height slip is 1.8 and is associated with a maximum depth of approximately 14 ft and a predicted volume of 4,500 yd^3. The analyses indicate the design final slope of 3H:1V satisfies the requirements of the acceptance criteria. Smaller-scale slips are possible under long-term static conditions; however, such occurrences are expected to be localised and not sizeable enough to dam the river.

4.3 SCOUR HOLE DESIGN

The material used to infill the scour hole will be a mix of E9/E9a - General Fill and Type CR1/CR2 - Concrete Rubble. Cover material will be a minimum of 6' of Type E9/E9b – General Fill and be comprised of scour hole cut material and/or forebay grading materials. Due to the anticipated method of material placement, the scour hole fill has been designed to have minimal compaction requirements. Placement requirements are detailed in Technical Specification 31 23 00 – Excavation and Fill Placement.

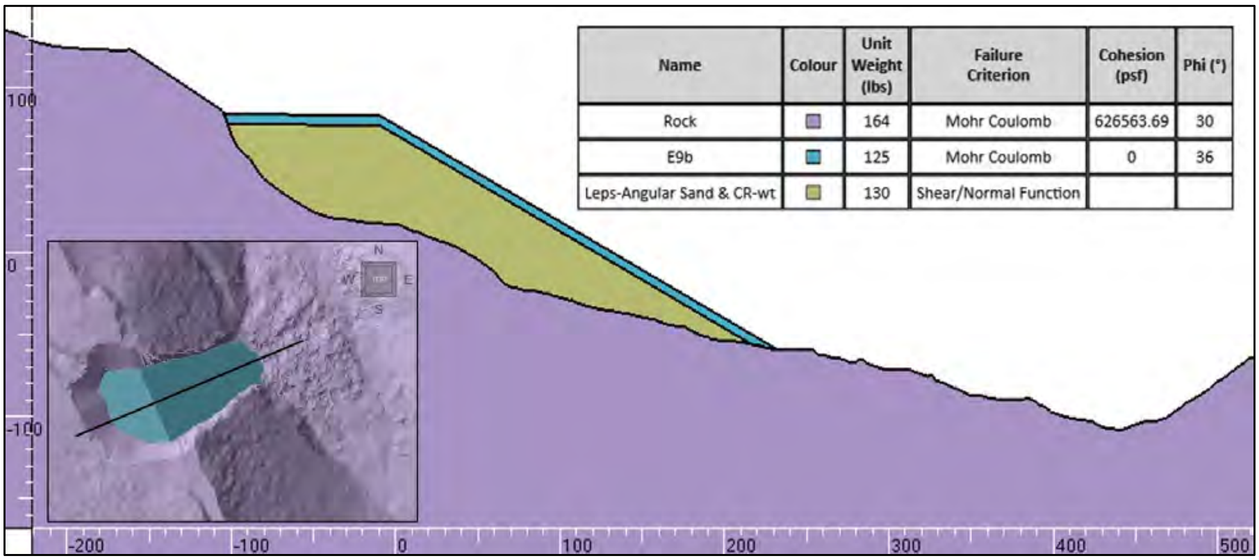
The scour hole design is shown on Drawings C1340 and C1341 and has the following design parameters:

- Maximum Slope: 1.7H:1V
- Crest Elevation: 3,728 ft
- Maximum Height: 140 ft

The stability of the scour hole design fill slope was assessed in a similar manner as the final excavated slope of the dam embankment. The same acceptance criteria and approach were used. The target FOS for the long-term static condition is 1.5 and 1.0 for determining the yield acceleration. Seismic displacements were estimated with semi-empirical methods (Makdisi and Seed, 1978; Bray and Travasarou, 2007) and a target of 2 ft. Pseudo-static analysis was precluded by estimating the seismic displacement. The short-term end-of-construction condition was not assessed since excess pore pressure generation and dissipation are not expected, given the free-draining nature of the design fills and the understanding of the foundation conditions. The creation of the scour hole and high-velocity nature of facility operations suggest fines would be mostly washed out of the accumulated coarse-grained erosion debris at the base of the hole.

The LEA was completed in three dimensions with Slide3 (Rocscience, 2020) and using both Spencer and GLE methods. A dry slope was assumed and the design seismic loading was defined by the STIDs (PacifiCorp, 2015a and 2015b).

The model geometry and material properties are shown on Figure 4.7. Material strengths were assumed based on the construction method and sequence. The Leps (1970) shear-normal function developed for the Angular Sand dataset was assumed for the fill strength but the equation was extrapolated to provide zero cohesion at zero normal stress. As a result, the analyses indicated the mobilized base friction angle was 38°. A constant value (36°) was assumed for the frictional strength of the 6-ft E9b cap based on gradation limits and compaction effort. Cohesive strength for both units was zero.



Name	Colour	Unit Weight (lbs)	Failure Criterion	Cohesion (psf)	Phi (°)
Rock		164	Mohr Coulomb	626563.69	30
E9b		125	Mohr Coulomb	0	36
Leps-Angular Sand & CR-wt		130	Shear/Normal Function		

NOTES:

1. SECTIONAL VIEW (LOOKING UPSTREAM) OF BLACK LINE IN INSET PLAN VIEW.

Figure 4.5 Scour Hole Model Geometry and Material Properties

The GLE method provided more conservative ky results and were again used for assessing the design. The yield acceleration slip search produced localized small-scale shallow slips with ky less than 0.17. A sizeable slip, extending from the fill crest to about mid-height, produced a ky of 0.17 and a displacement estimate greater than 2 ft. The maximum depth of the associated slip is about 17 ft and involves a volume of approximately 4,500 yd^3. The estimated volume of such a slip is not expected to dam the river, given it is less than 10% of the overall volume of the design fill and erosion of the scour hole was not known to have dammed the river or impeded its flow in the past. A full-height (30-ft deep) slip with a ky of 0.18 was estimated to displace less than 2 ft and involves a volume of approximately 11,500 yd^3. The results of these two slips are shown on Figure 4.8.

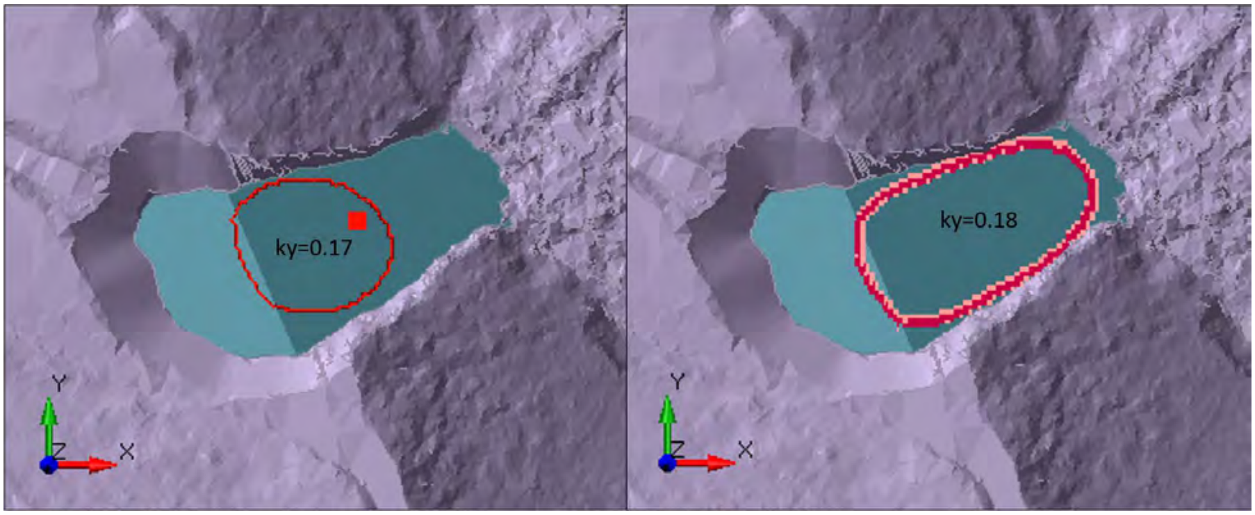
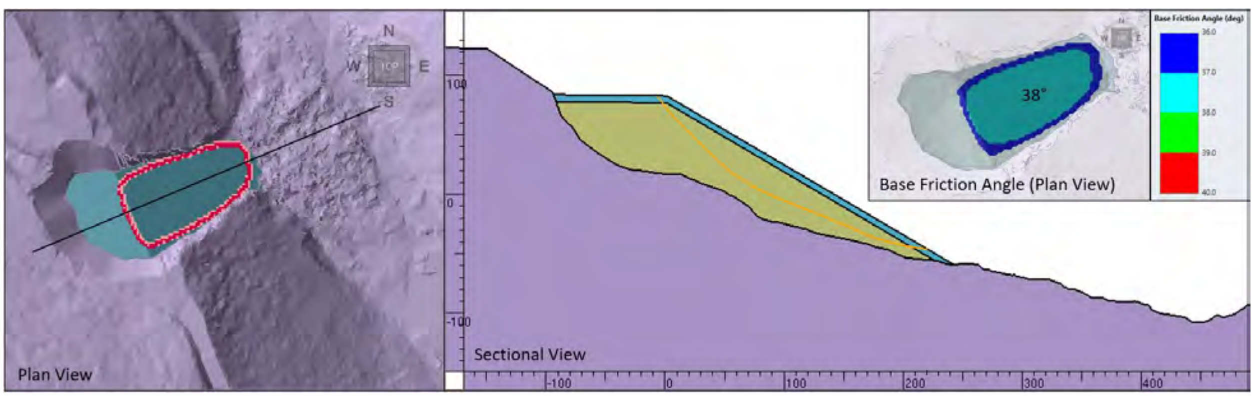


Figure 4.6 Scour Hole Yield Acceleration Results

The static result for a full-height slip marginally satisfies the target FOS of 1.5 for dry conditions, with both Spencer and GLE methods. The critical slip is predicted to involve a volume of approximately 13,500 yd^3 with a maximum depth of roughly 35 ft. The critical slip of the static analysis is shown on Figure 4.9. Search results indicate localized smaller-scale slips could occur but are found within the 6-ft E9b cap material.



NOTES:

1. SECTIONAL VIEW (LOOKING UPSTREAM) OF CRITICAL SLIP (YELLOW LINE) SHOWN IN THE RIGHT IMAGE. BLACK LINE IN PLAN VIEW (LEFT IMAGE) SHOWS LOCATION OF SECTION LINE.
2. BASE FRICTION ANGLE CONTOURS SHOWN IN INSET. ZERO COHESIVE STRENGTH WAS ASSIGNED.

Figure 4.7 Scour Hole Static Stability Results

4.4 POWER CANAL AND PENSTOCK ACCESS ROAD SLOPE HAZARDS

The power canal slope terrain hazard assessment identifies hazards along the alignment. It is included as part of the Geotechnical Data Report (VA103-640/1-2). The existing power canal plan and typical sections are shown on Drawings C1320, C1321, and C1323.

The penstock access road slope terrain hazard assessment has also been completed. The extent of anticipated access road stabilization is limited to the lower penstock access roads shown on Drawing

C1512. The access road slope terrain hazard assessment is included as part of the Geotechnical Data Report (VA103-640/1-2).

4.5 BORROW AREAS

Borrow areas may be required at the intake and the forebay area to provide backfilling material. The intake borrow areas are to be evaluated during construction and may be used as erosion protection materials for the Final River Channel as shown on Drawings C1210 and C1230. The forebay borrow source proposed grading is required to cover the forebay concrete structure left in place and provide scour hole fill materials and is shown on Drawings C1334 and C1335. The grading is designed to convey direct precipitation to the scour hole drainage swale.

4.6 DISPOSAL SITES

4.6.1 FOUNDATION PREPARATION

Geotechnical data and investigations are presented in the Geotechnical Data Report (VA103-640/1-2). On site investigations have been completed for the original upland disposal site. Foundation conditions for the river bank disposal sites require inspection after drawdown, under dry conditions.

Foundation preparation of staging and disposal sites will consist of removing and stockpiling topsoil and organic and soft materials away from the disposal site. Topsoil and organic materials may be redistributed on the disposal site slopes following embankment and historic cofferdam excavation.

4.6.2 DISPOSAL SITE DESIGN

Two concepts were considered for siting the permanent disposal areas. The preferred concept is along the left river bank. An additional disposal site on the right river bank has been designed for use if the left bank disposal site reaches capacity.

4.6.2.1 RIVER BANK DISPOSAL SITES

The proposed disposal areas are shown on Drawings C1240 and C1241. Disposal areas are designed with stable permanent slopes and suitable drainage requirements.

The disposal site fill materials will be comprised of dam embankment, historic cofferdam, and remaining sediment excavation materials, expected to meet the, E9/E9a – General Fill and E10 - Random Fill material specifications. The disposal sites that are considered for this excavation are described as follows:

- The Left Bank Disposal Site which will hold the majority of the excavated materials and include an area of cover fill on the remainder of the spillway and intake structure concrete left in place.
- The Alternative Right Bank Disposal Site which, if used, will be primarily comprised of excavated material from the final cofferdam breach.

The material placed in the disposal sites will be track packed and graded with a bulldozer to meet the requirements of the technical specifications. The disposal sites will be graded to promote surface drainage towards the Klamath River. Parameters for the Left Bank Disposal Area are as follows:

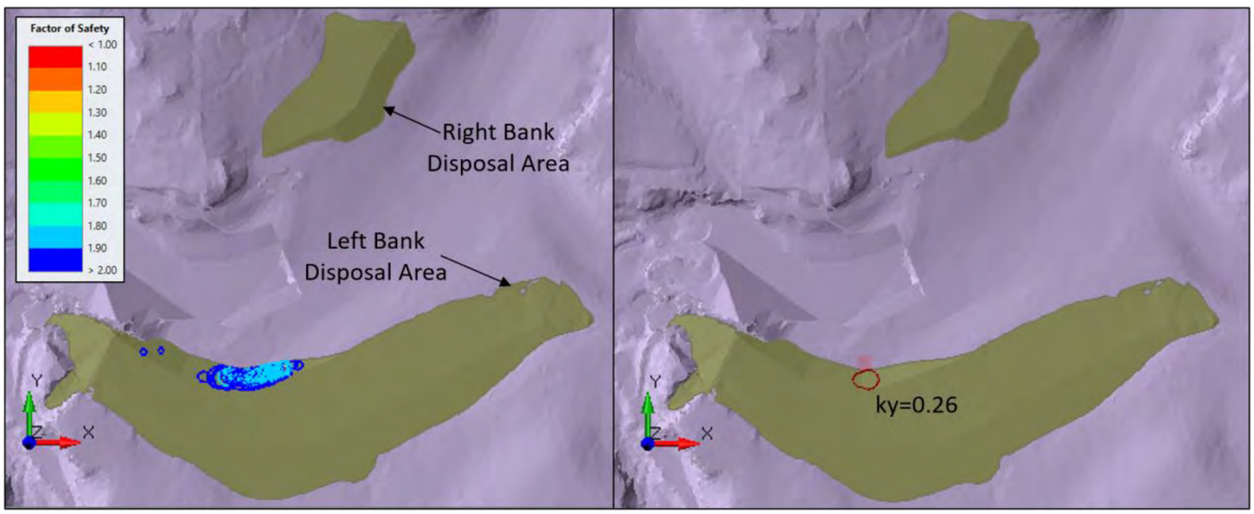
- Maximum Slope: 3.5H:1V
- Crest Elevation: 3,800 ft

- Maximum Height: 35 ft

Parameters for the Alternative Right Bank Disposal Area are as follows:

- Maximum Slope: 5H:1V
- Crest Elevation: 3,794 ft
- Maximum Height: 18 ft

Stability of the two river bank disposal areas were assessed by LEA, in three dimensions with Slide3 (Rocscience, 2020) and using both Spencer and GLE methods. The target FOS for the long-term static condition is 1.5 and 1.0 for determining the yield acceleration. Seismic displacements were estimated with semi-empirical methods (Makdisi and Seed, 1978; Bray and Travasarou, 2007) and a target of 2 ft. Pseudo-static analyses was completed with a 20% strength reduction for 50% MCE. The short-term end-of-construction condition was not assessed since excess pore pressure generation and dissipation are not expected, given the free-draining nature of the design fills and the understanding of the foundation conditions. The design seismic loading was defined by the STIDs (PacifiCorp, 2015a and 2015b). The analysis model and results of the disposal areas are shown on Figure 4.10.



NOTES:

1. STATIC SEARCH RESULTS SHOWN IN LEFT, ONLY SLIPS WITH FOS < 2.0 ARE SHOWN.
2. YIELD ACCELERATION RESULT SHOWN IN RIGHT.

Figure 4.8 Disposal Sites Stability Results

The disposal fill is E9 and the frictional strength was assigned 32° based on design gradation limits and compaction requirements. Zero cohesive strength and dry piezometric conditions were assumed. The right bank disposal area satisfies the requirements for static and pseudo-static stability. The left bank disposal area satisfies static stability but not pseudo-static stability. Seismic displacements were estimated for a yield acceleration of 0.26 and less than 2 ft of movement was predicted.

4.6.2.2 ALTERNATIVE UPLAND DISPOSAL SITE

An alternative to the river bank disposal sites is the original borrow source area that was developed for dam construction and the location of the disposal site considered during preliminary design. Preliminary analyses indicate slope stability achieves a factor of safety of 1.5 for dry static conditions.

Attachments:
1 – J.C. Boyle Facility Simulated Drawdown

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J.C. BOYLE FACILITY SIMULATED DRAWDOWN

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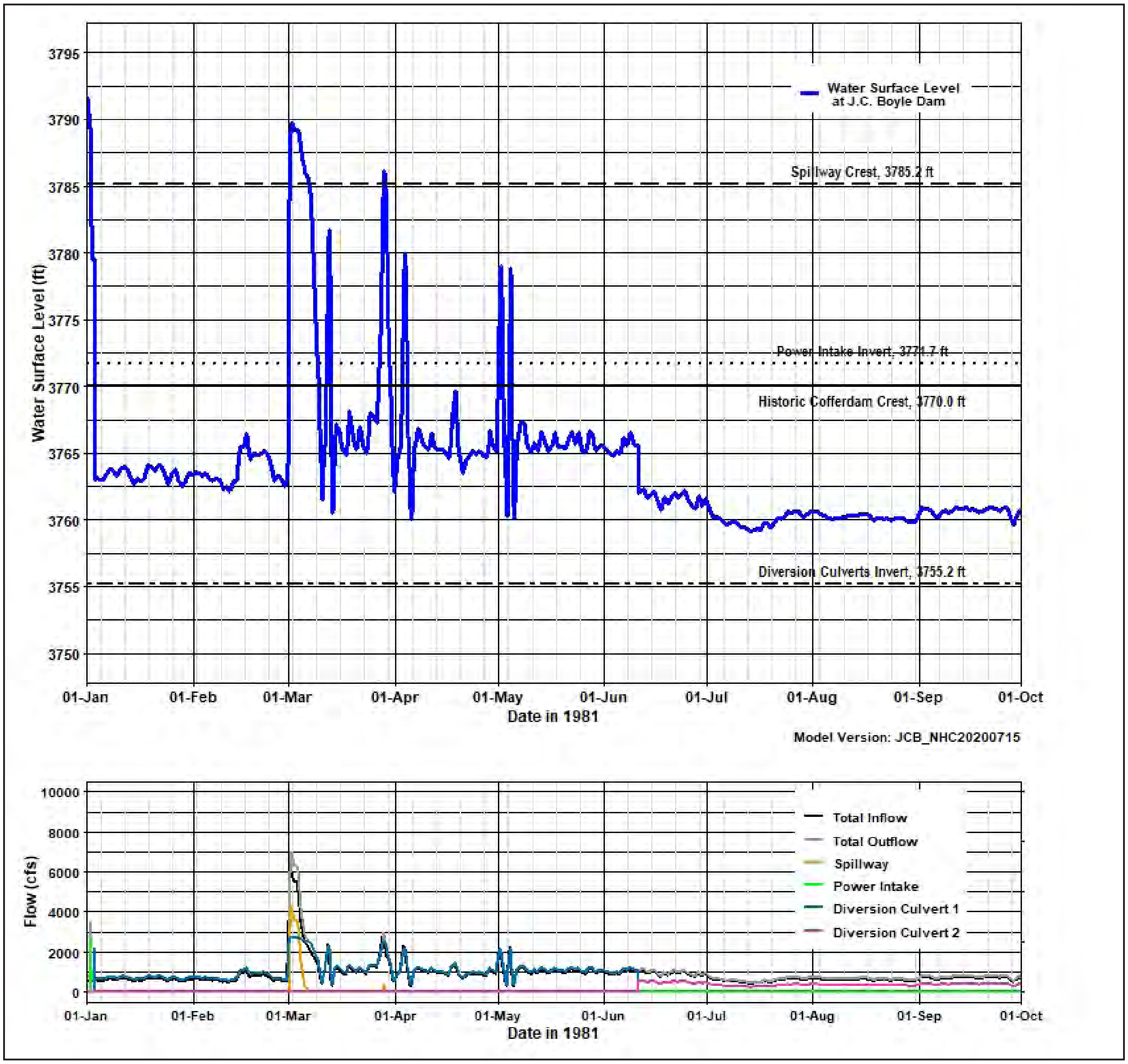

Figure 1 - J.C. Boyle Facility Simulated Drawdown - Year 1981

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J.C. BOYLE FACILITY SIMULATED DRAWDOWN

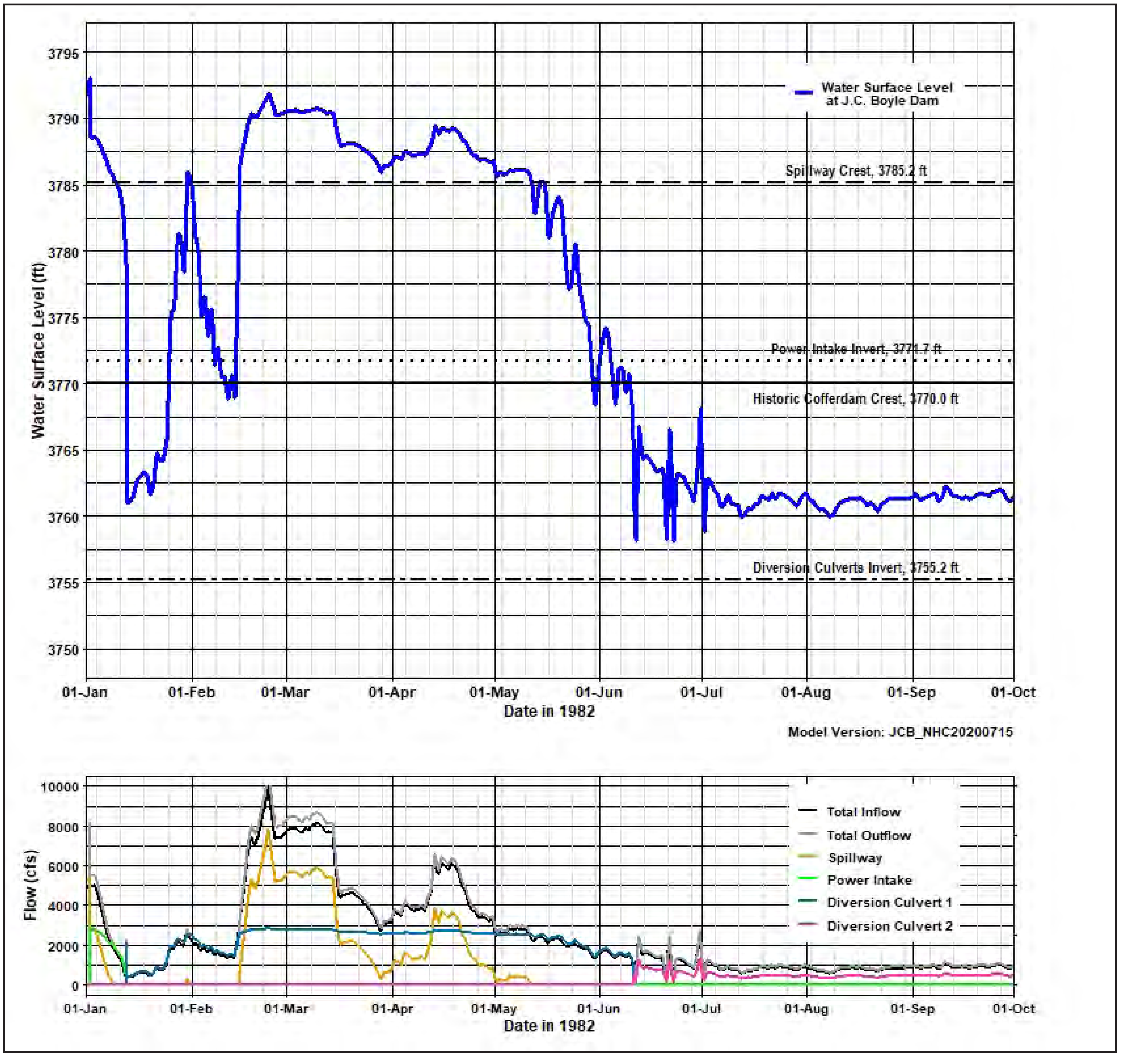

Figure 2 - J.C. Boyle Facility Simulated Drawdown - Year 1982

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J.C. BOYLE FACILITY SIMULATED DRAWDOWN

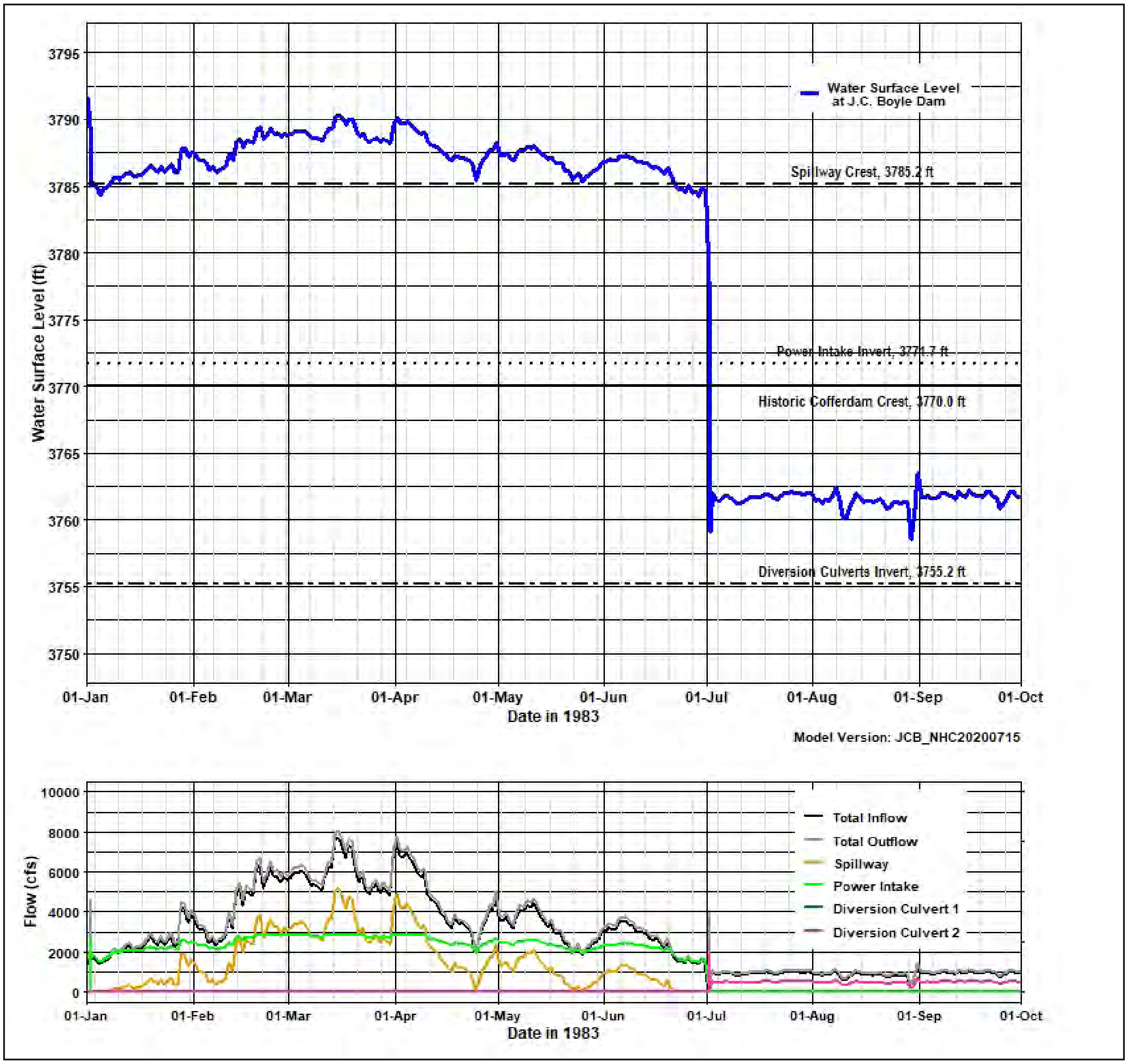

Figure 3 - J.C. Boyle Facility Simulated Drawdown - Year 1983

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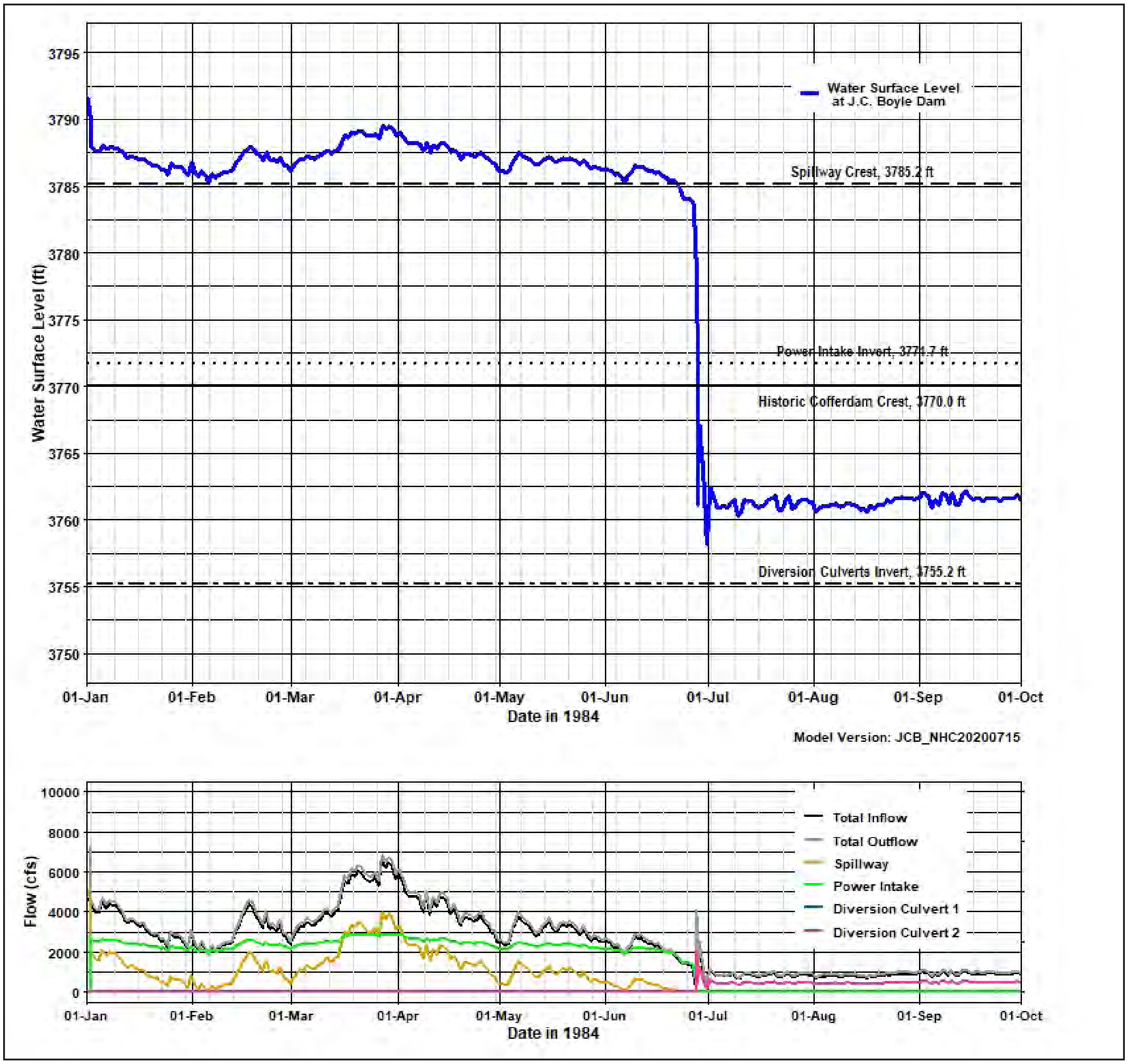

Figure 4 - J.C. Boyle Facility Simulated Drawdown - Year 1984

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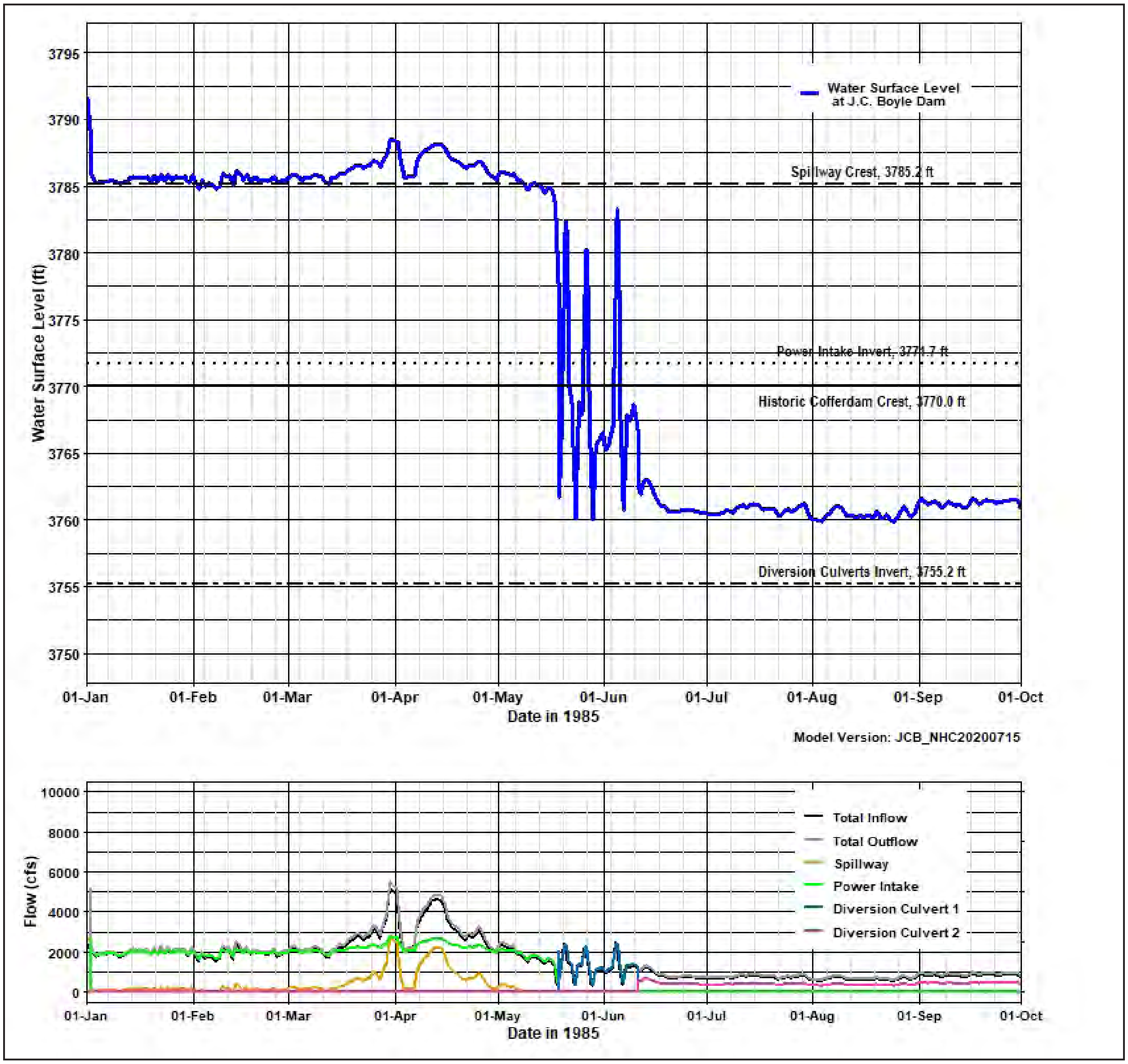

Figure 5 - J.C. Boyle Facility Simulated Drawdown - Year 1985

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J.C. BOYLE FACILITY SIMULATED DRAWDOWN

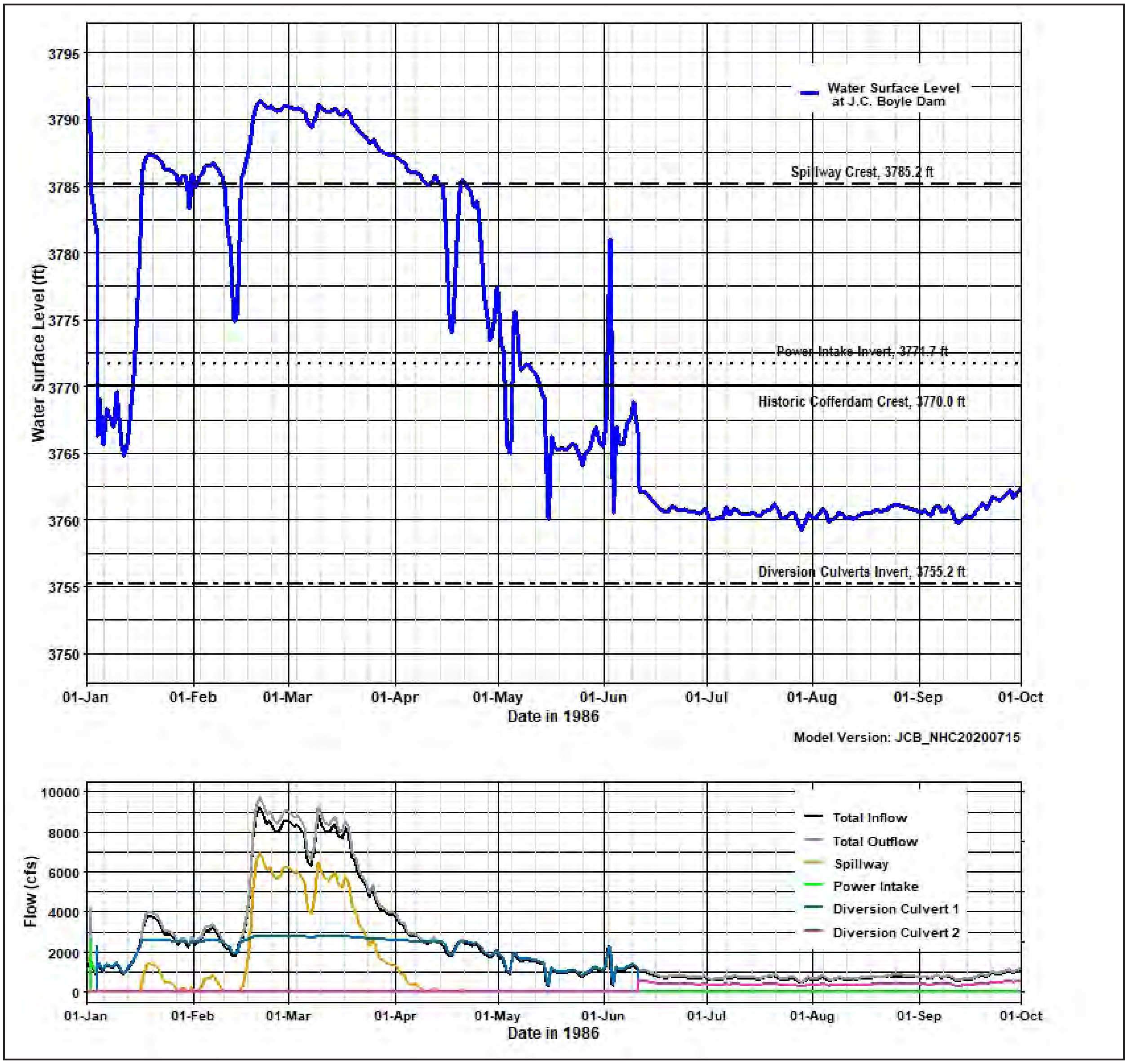

Figure 6 - J.C. Boyle Facility Simulated Drawdown - Year 1986

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J.C. BOYLE FACILITY SIMULATED DRAWDOWN

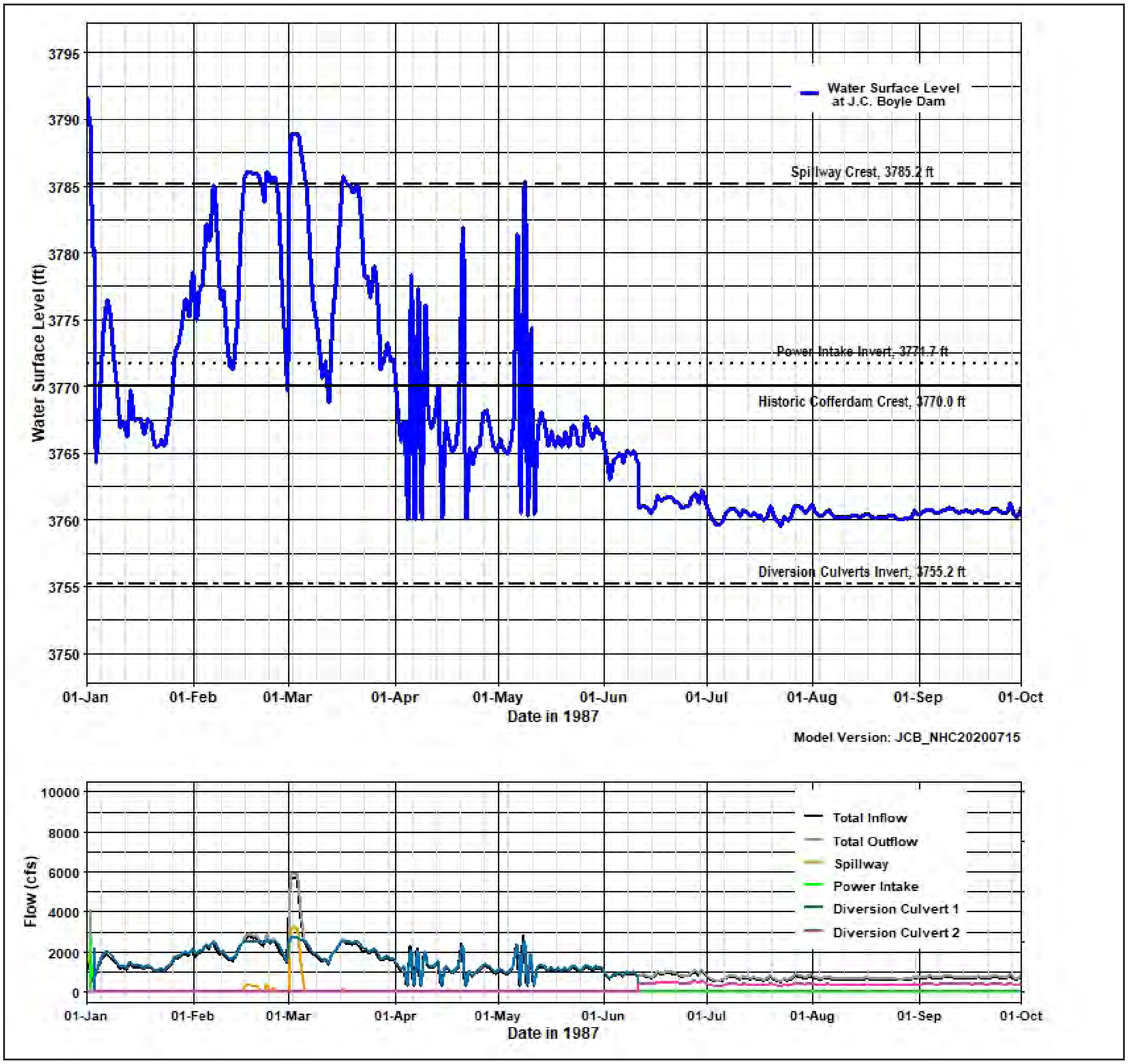

Figure 7 - J.C. Boyle Facility Simulated Drawdown - Year 1987

July 29, 2020

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J.C. BOYLE FACILITY SIMULATED DRAWDOWN

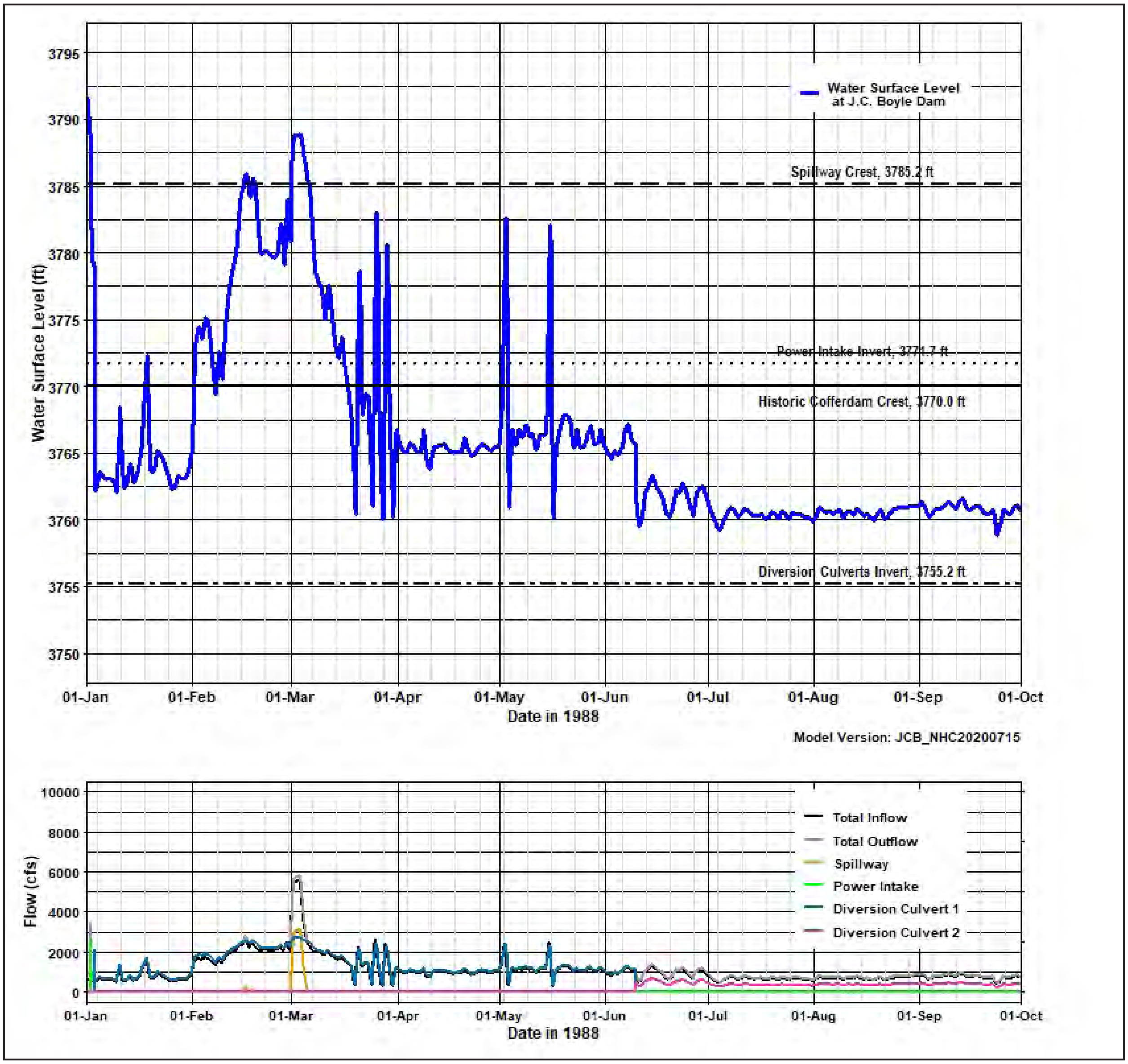

Figure 8 - J.C. Boyle Facility Simulated Drawdown - Year 1988

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J.C. BOYLE FACILITY SIMULATED DRAWDOWN

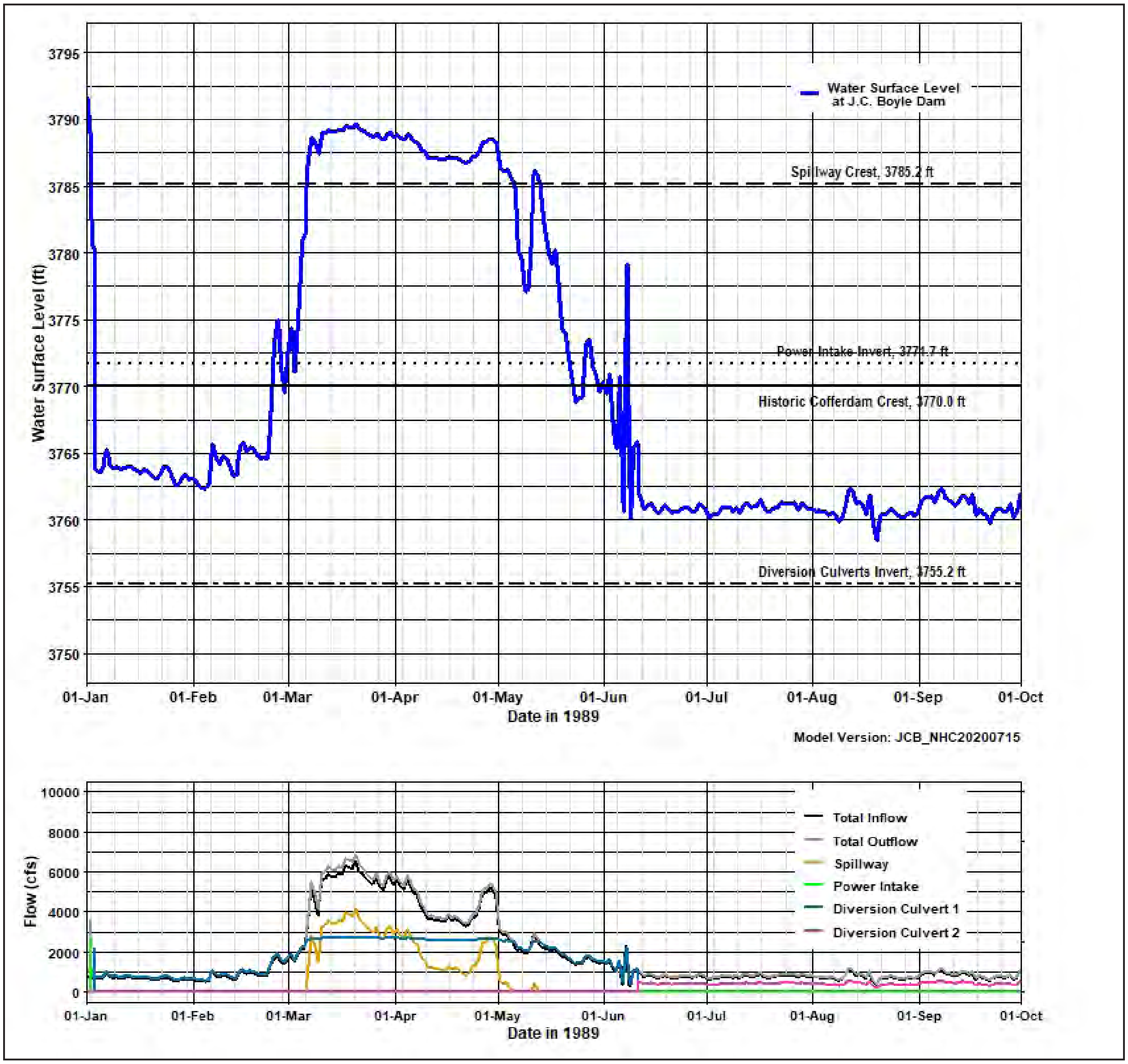

Figure 9 - J.C. Boyle Facility Simulated Drawdown - Year 1989

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J.C. BOYLE FACILITY SIMULATED DRAWDOWN

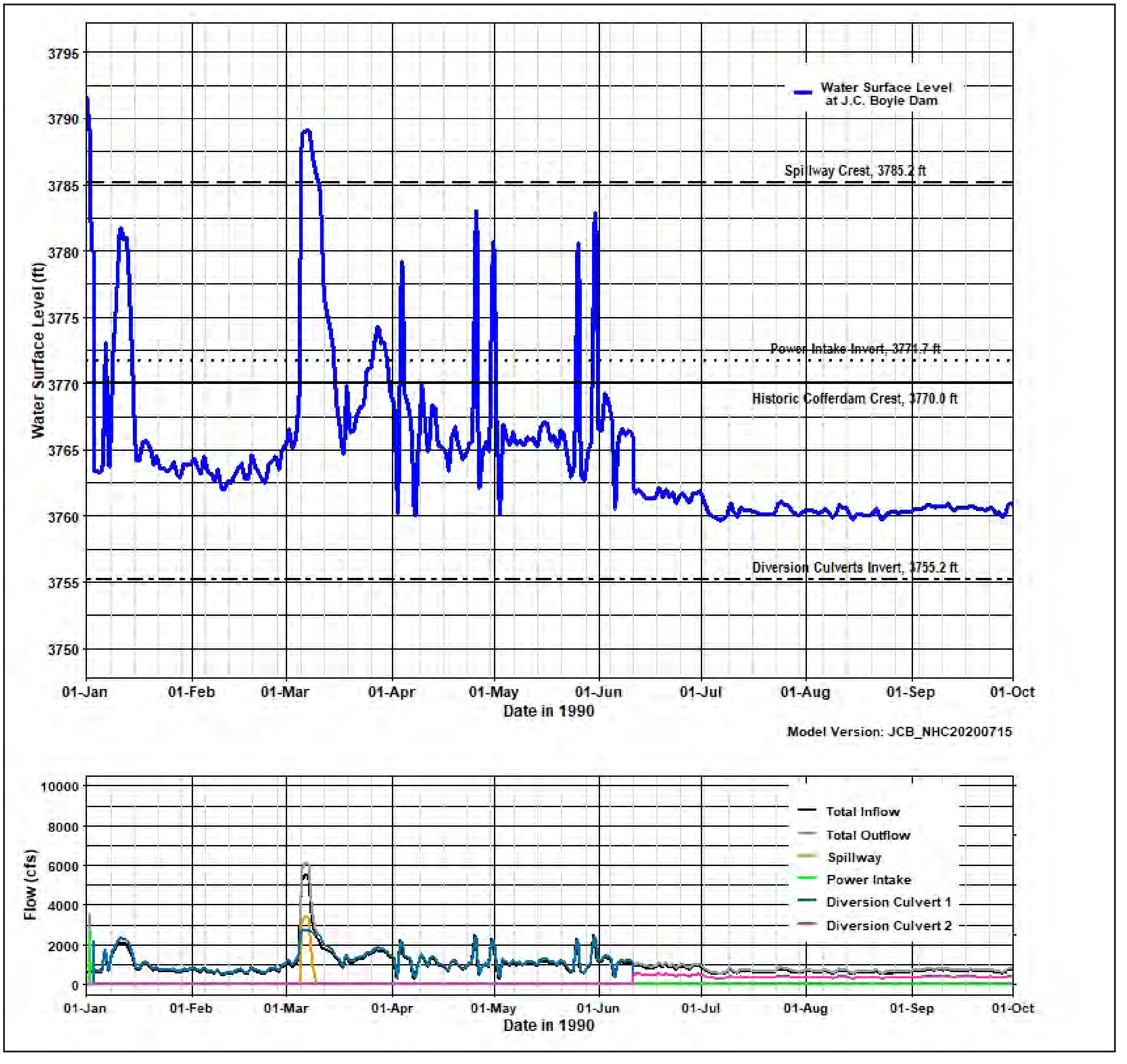

Figure 10 - J.C. Boyle Facility Simulated Drawdown - Year 1990

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J.C. BOYLE FACILITY SIMULATED DRAWDOWN

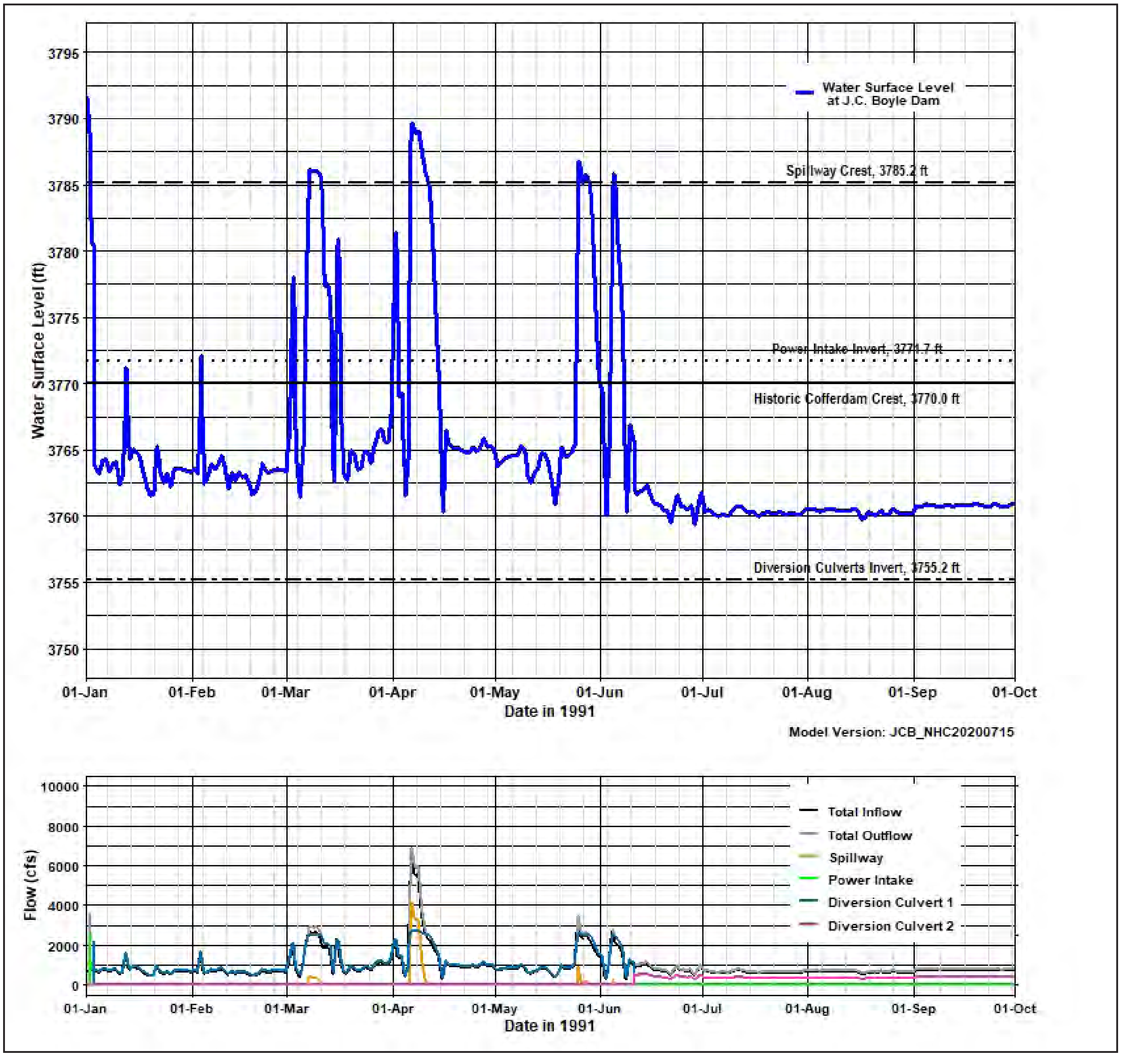

Figure 11 - J.C. Boyle Facility Simulated Drawdown - Year 1991

July 29, 2020

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J.C. BOYLE FACILITY SIMULATED DRAWDOWN

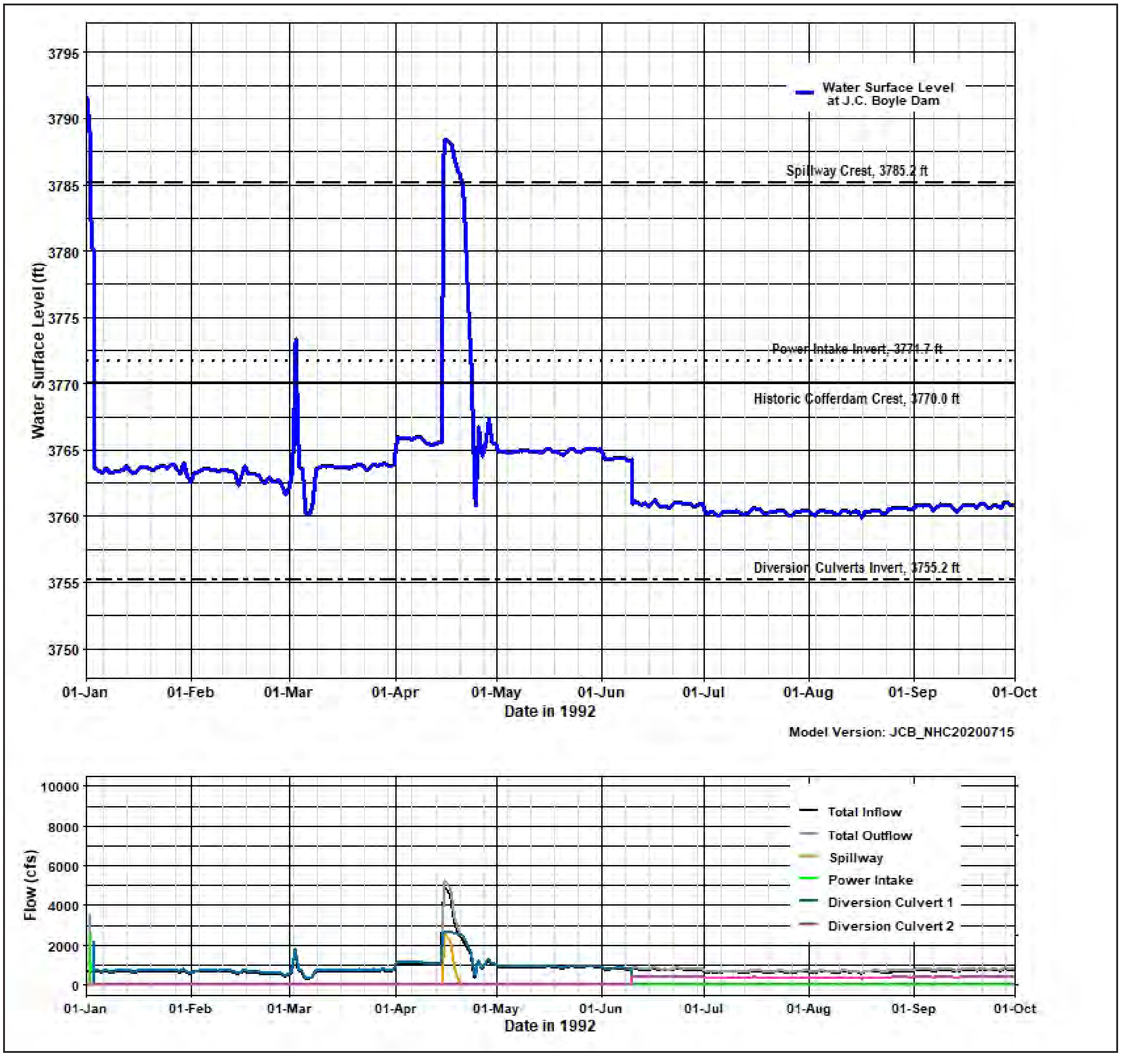

Figure 12 - J.C. Boyle Facility Simulated Drawdown - Year 1992

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J.C. BOYLE FACILITY SIMULATED DRAWDOWN

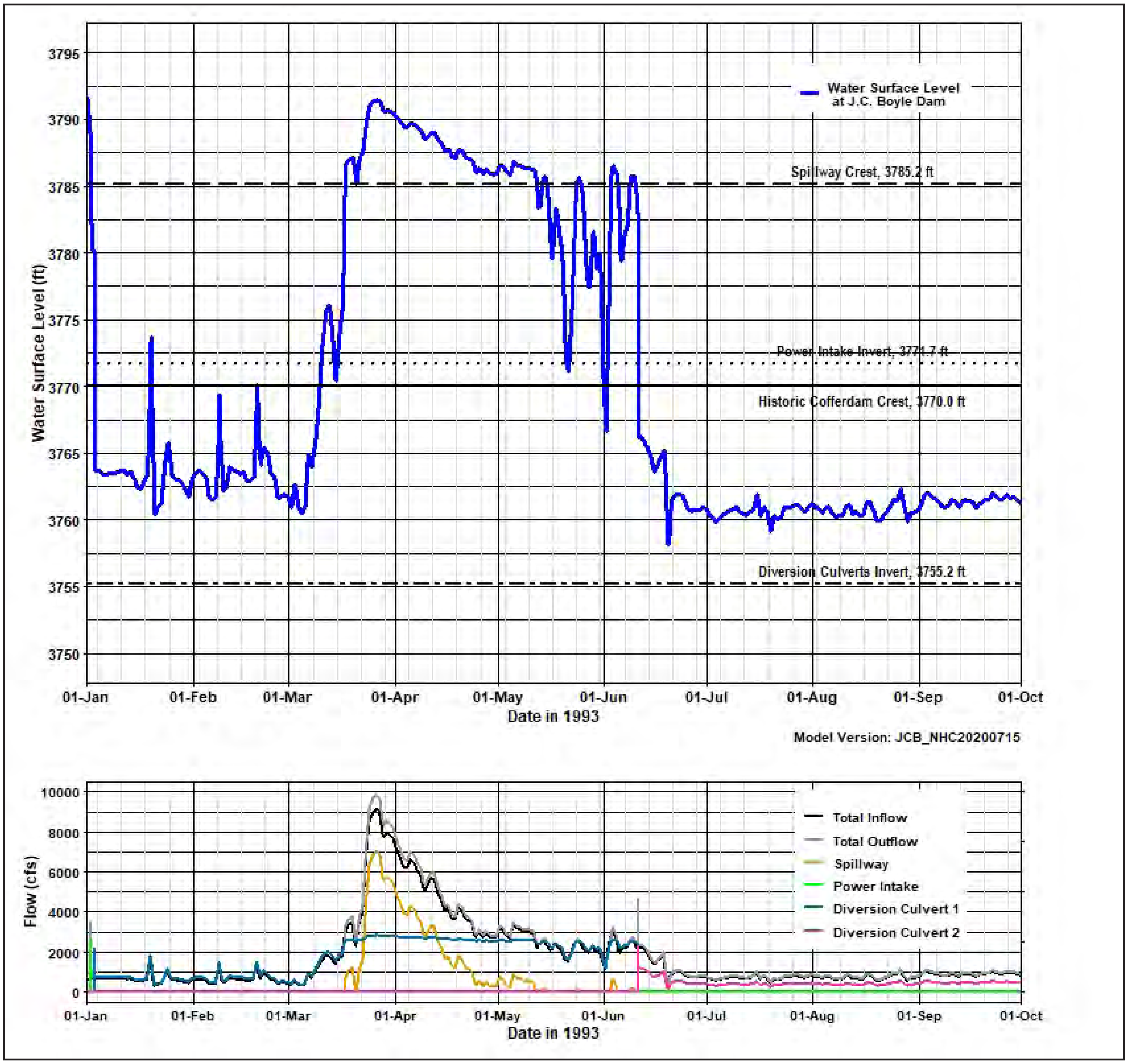

Figure 13 - J.C. Boyle Facility Simulated Drawdown - Year 1993

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J.C. BOYLE FACILITY SIMULATED DRAWDOWN

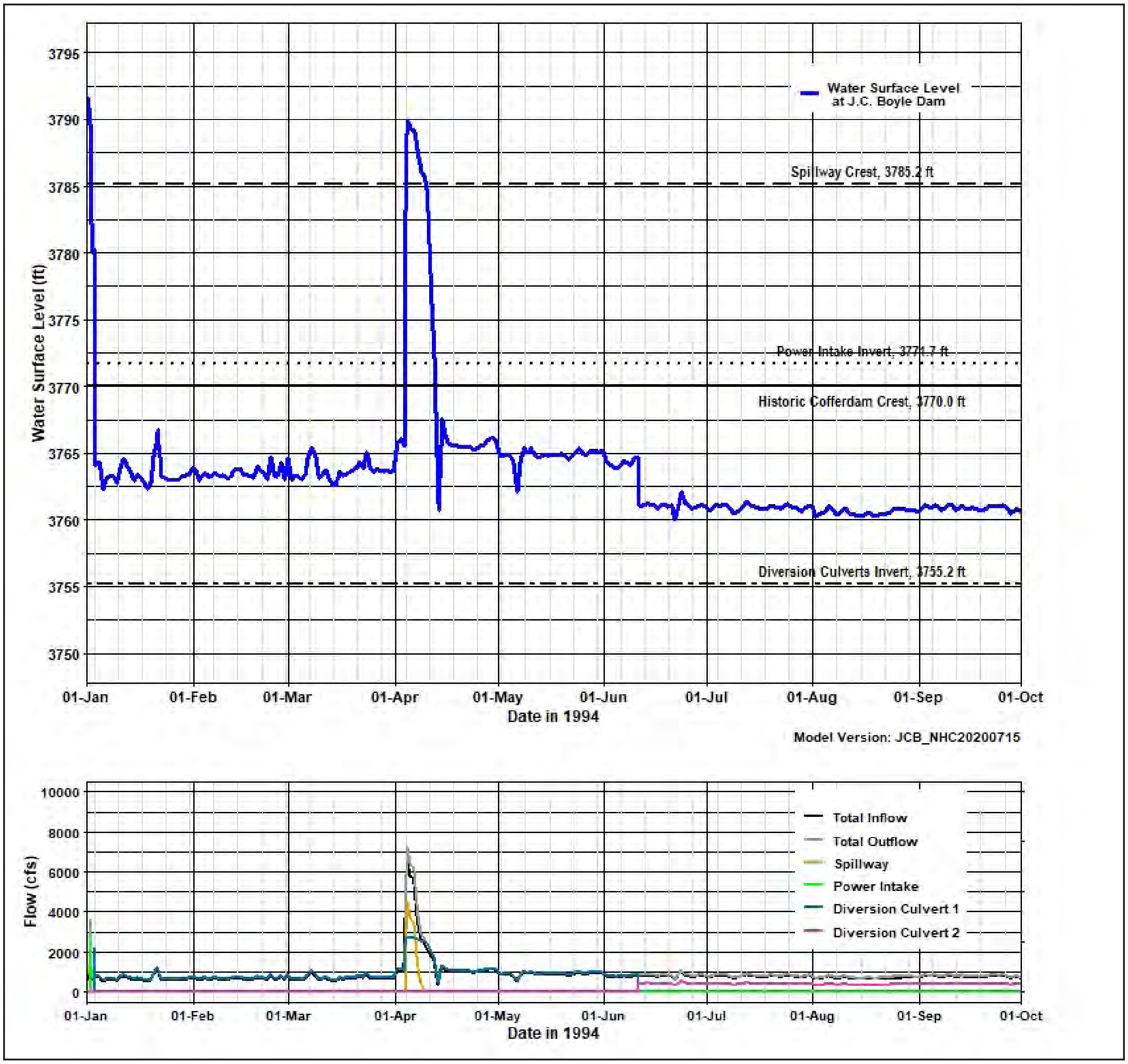

Figure 14 - J.C. Boyle Facility Simulated Drawdown - Year 1994

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J.C. BOYLE FACILITY SIMULATED DRAWDOWN

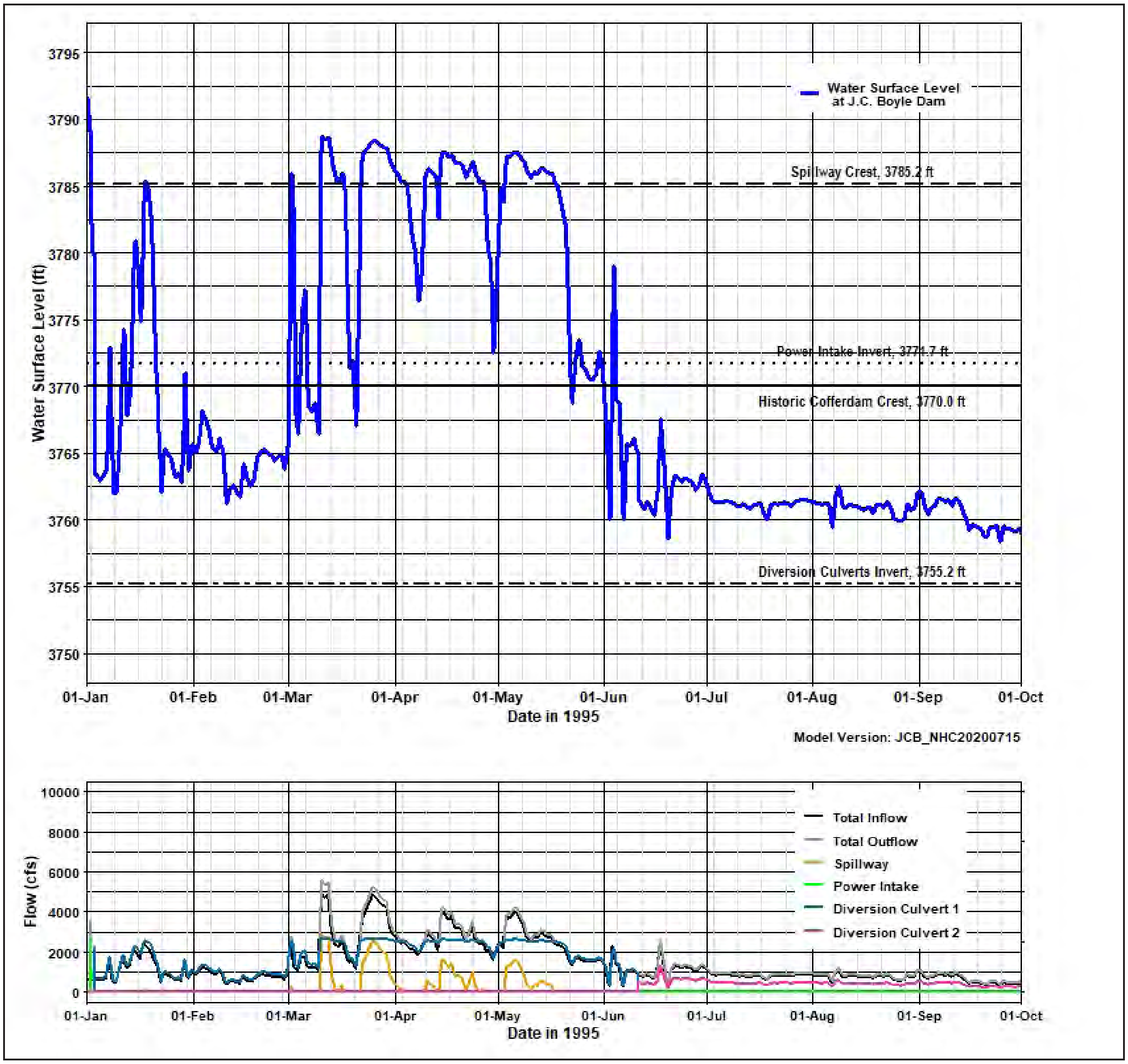

Figure 15 - J.C. Boyle Facility Simulated Drawdown - Year 1995

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J.C. BOYLE FACILITY SIMULATED DRAWDOWN

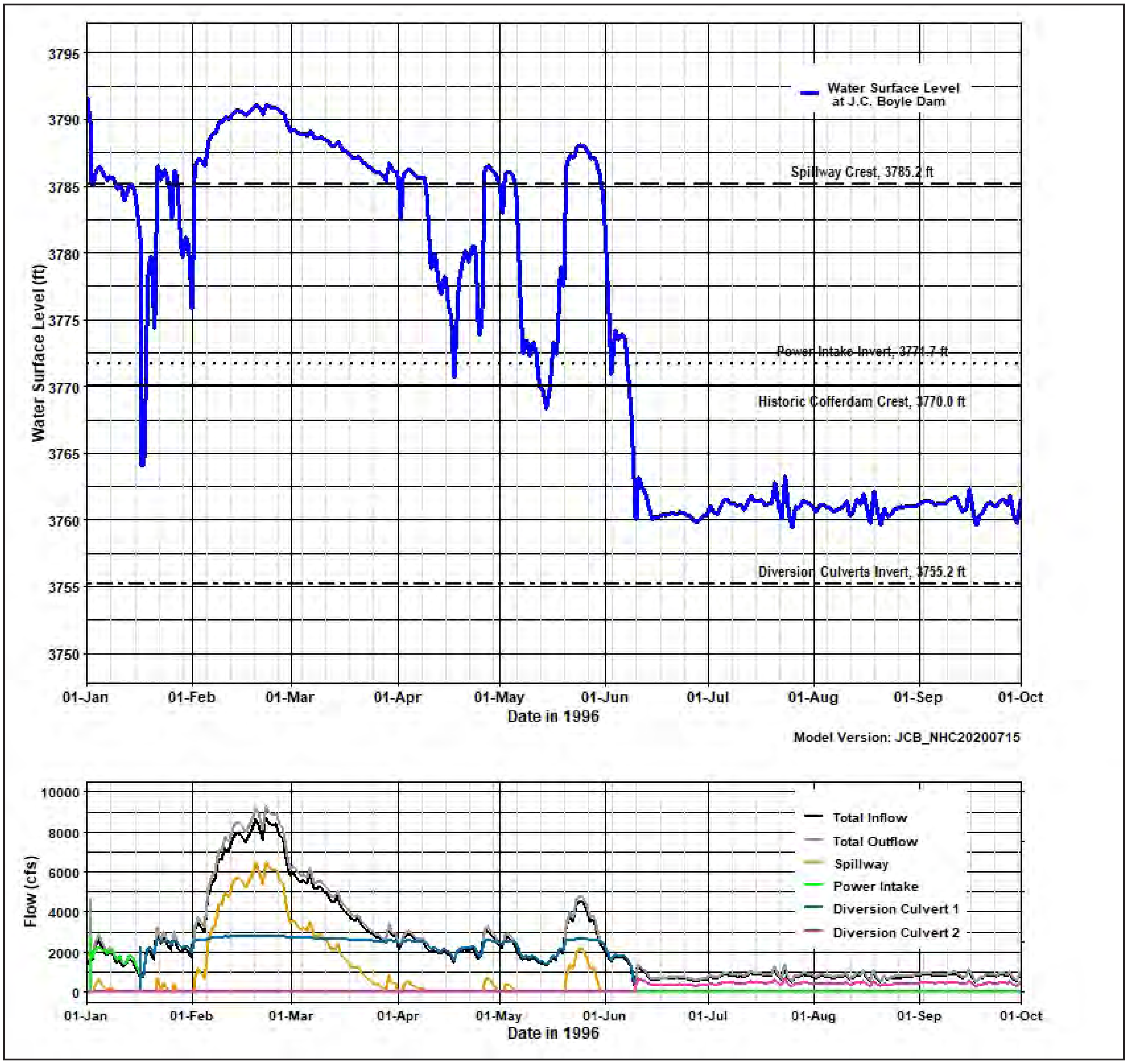

Figure 16 - J.C. Boyle Facility Simulated Drawdown - Year 1996

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J.C. BOYLE FACILITY SIMULATED DRAWDOWN

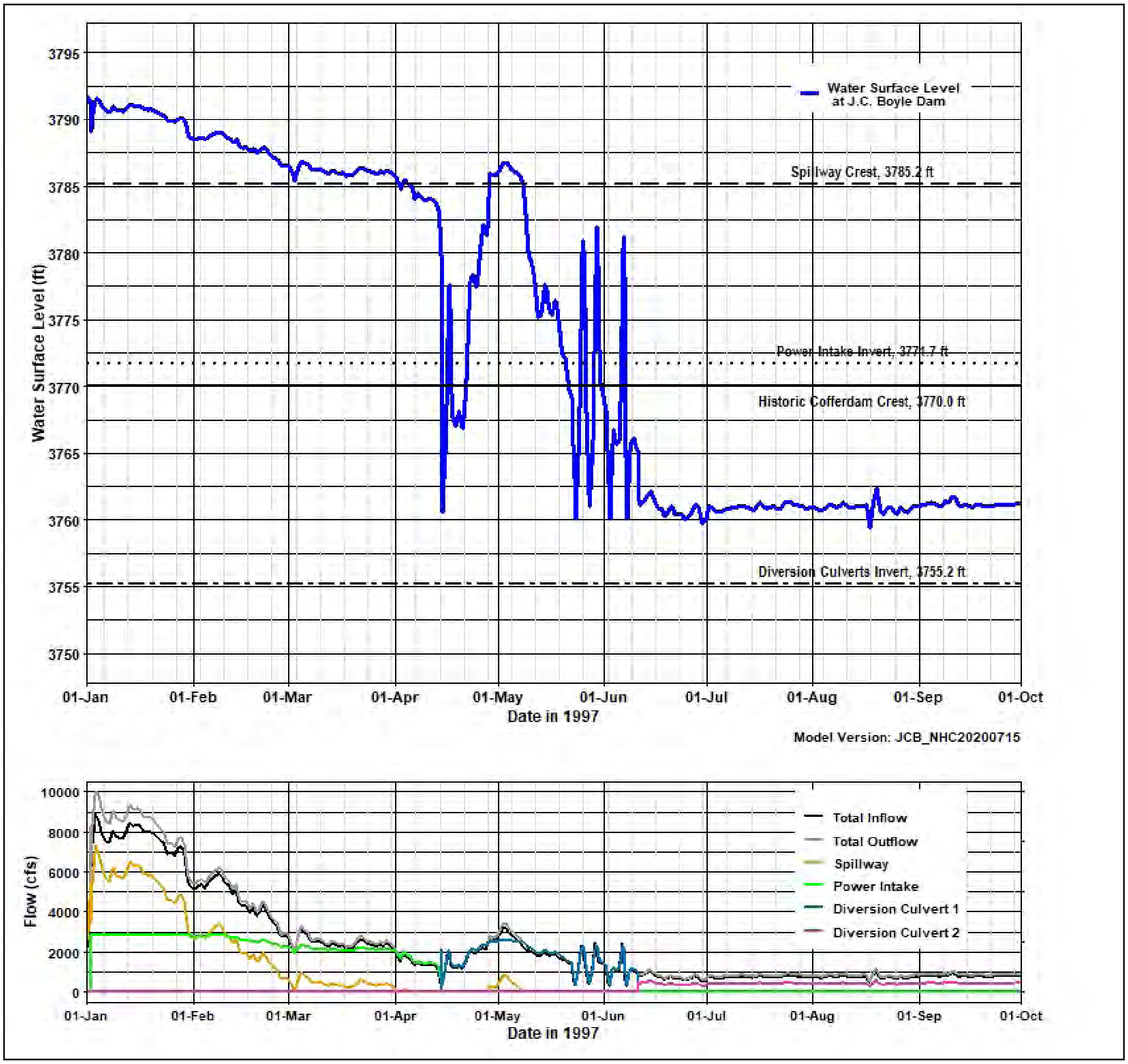

Figure 17 - J.C. Boyle Facility Simulated Drawdown - Year 1997

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J.C. BOYLE FACILITY SIMULATED DRAWDOWN

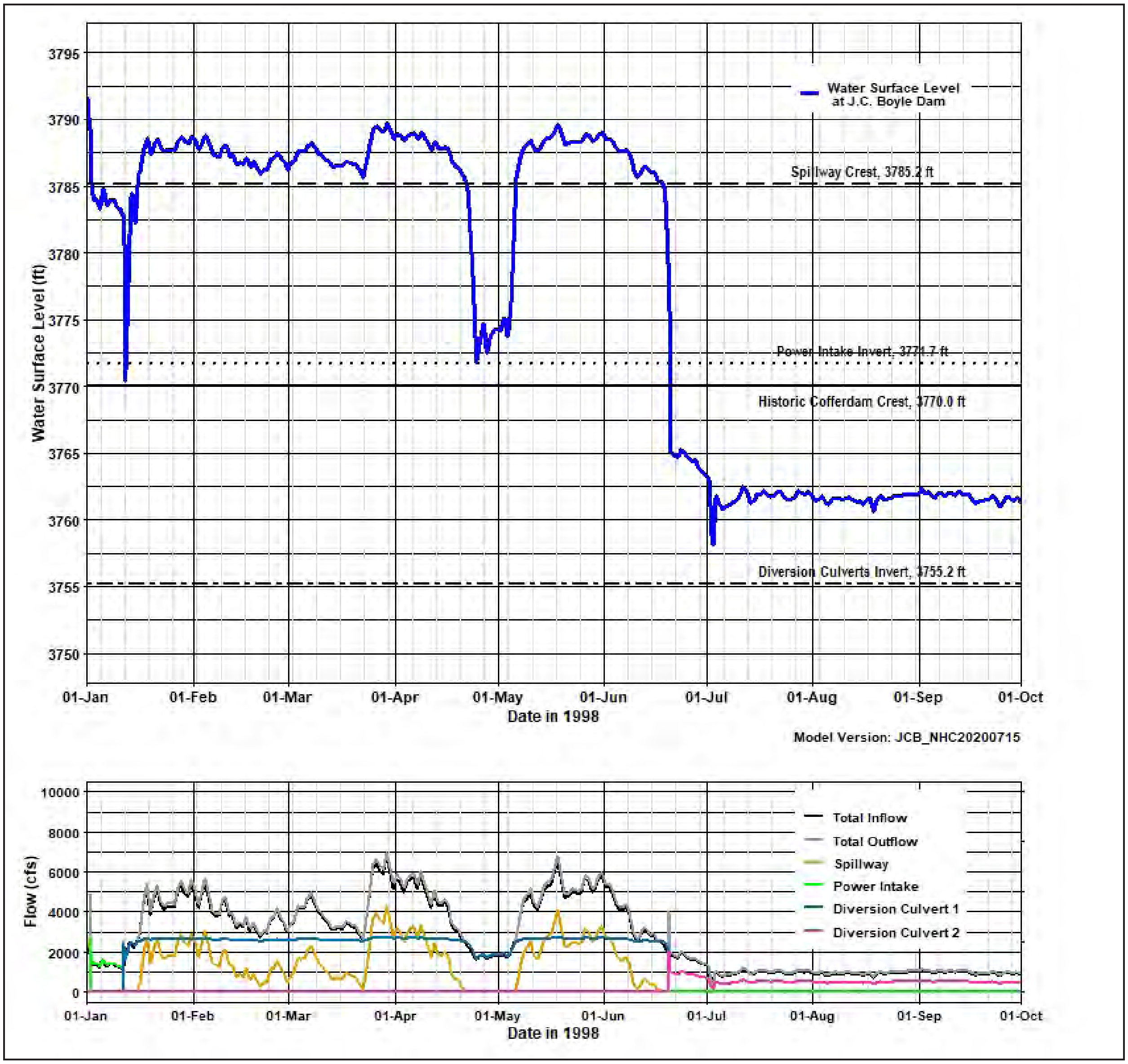

Figure 18 - J.C. Boyle Facility Simulated Drawdown - Year 1998

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J.C. BOYLE FACILITY
SIMULATED DRAWDOWN

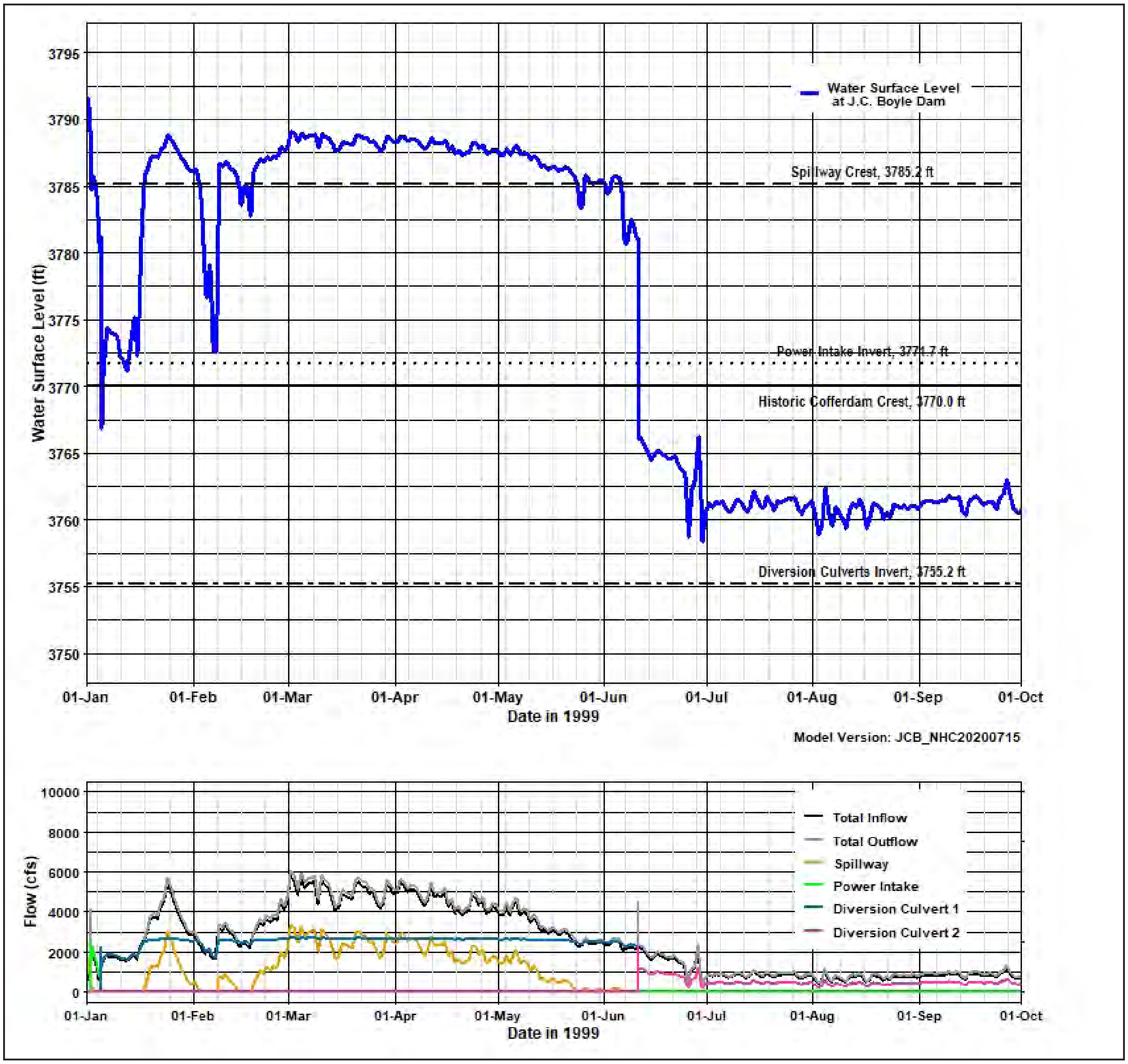

Figure 19 - J.C. Boyle Facility Simulated Drawdown - Year 1999

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J.C. BOYLE FACILITY SIMULATED DRAWDOWN

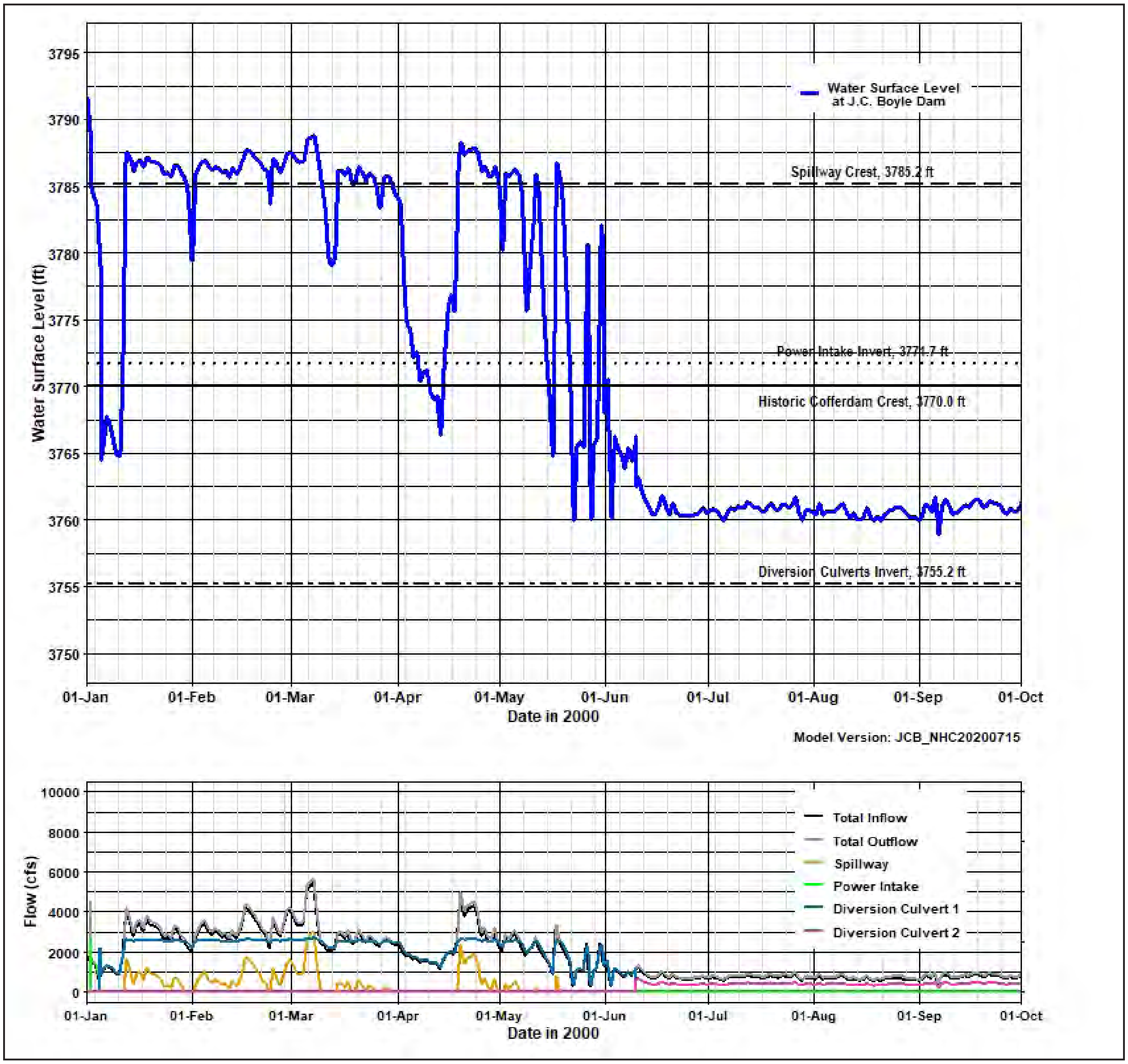

Figure 20 - J.C. Boyle Facility Simulated Drawdown - Year 2000

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J.C. BOYLE FACILITY SIMULATED DRAWDOWN

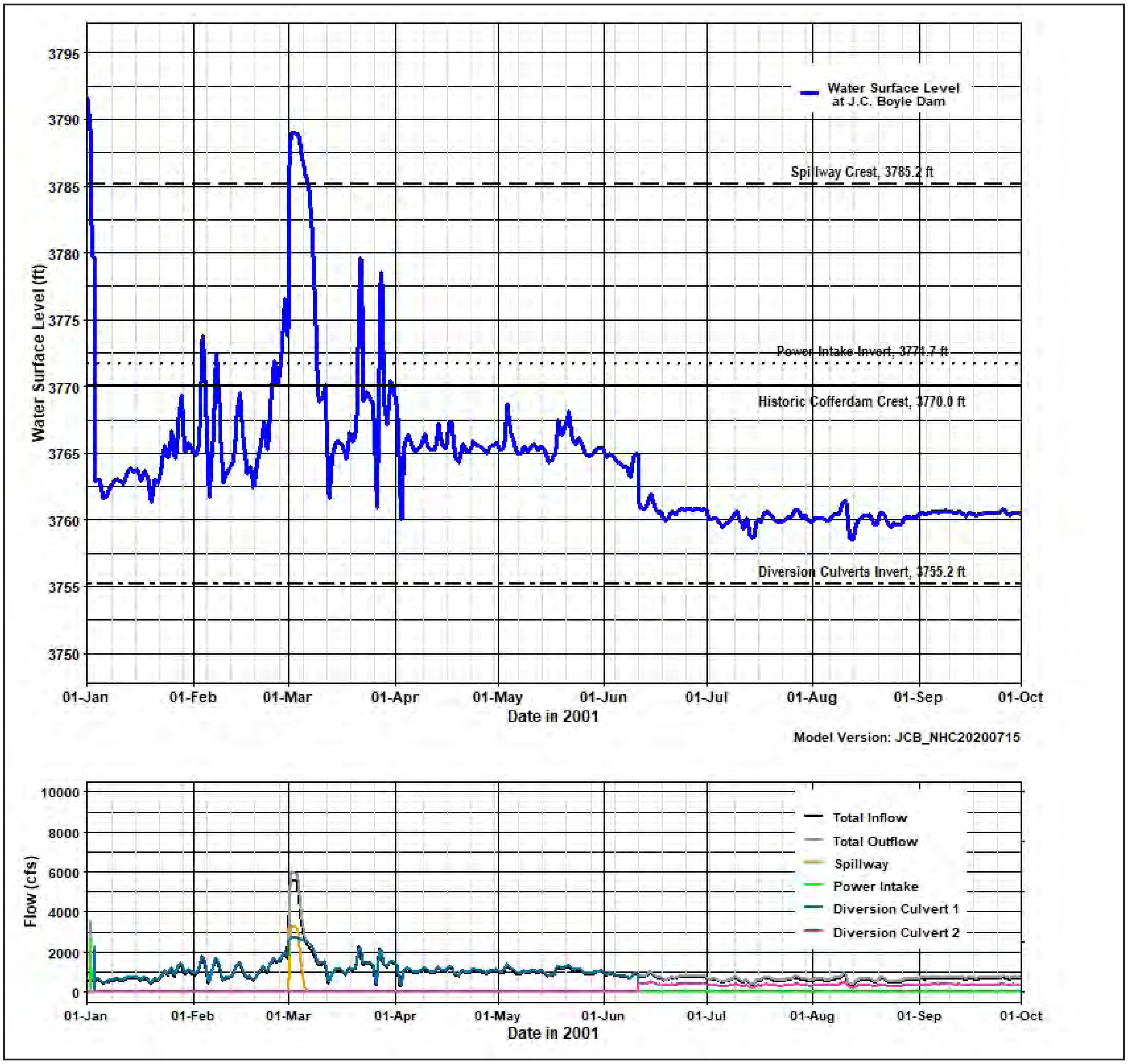

Figure 21 - J.C. Boyle Facility Simulated Drawdown - Year 2001

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J.C. BOYLE FACILITY SIMULATED DRAWDOWN

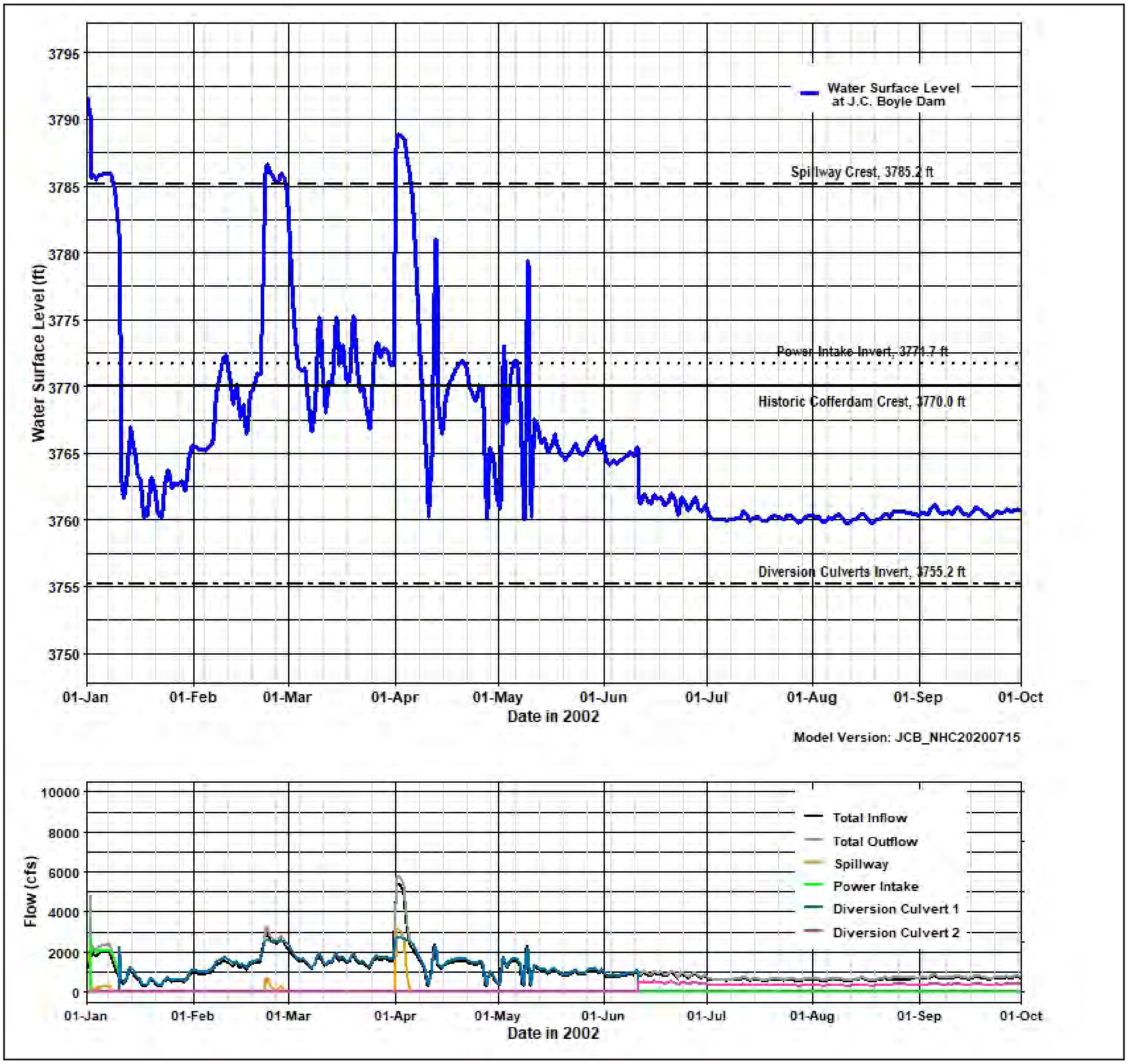

Figure 22 - J.C. Boyle Facility Simulated Drawdown - Year 2002

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J.C. BOYLE FACILITY SIMULATED DRAWDOWN

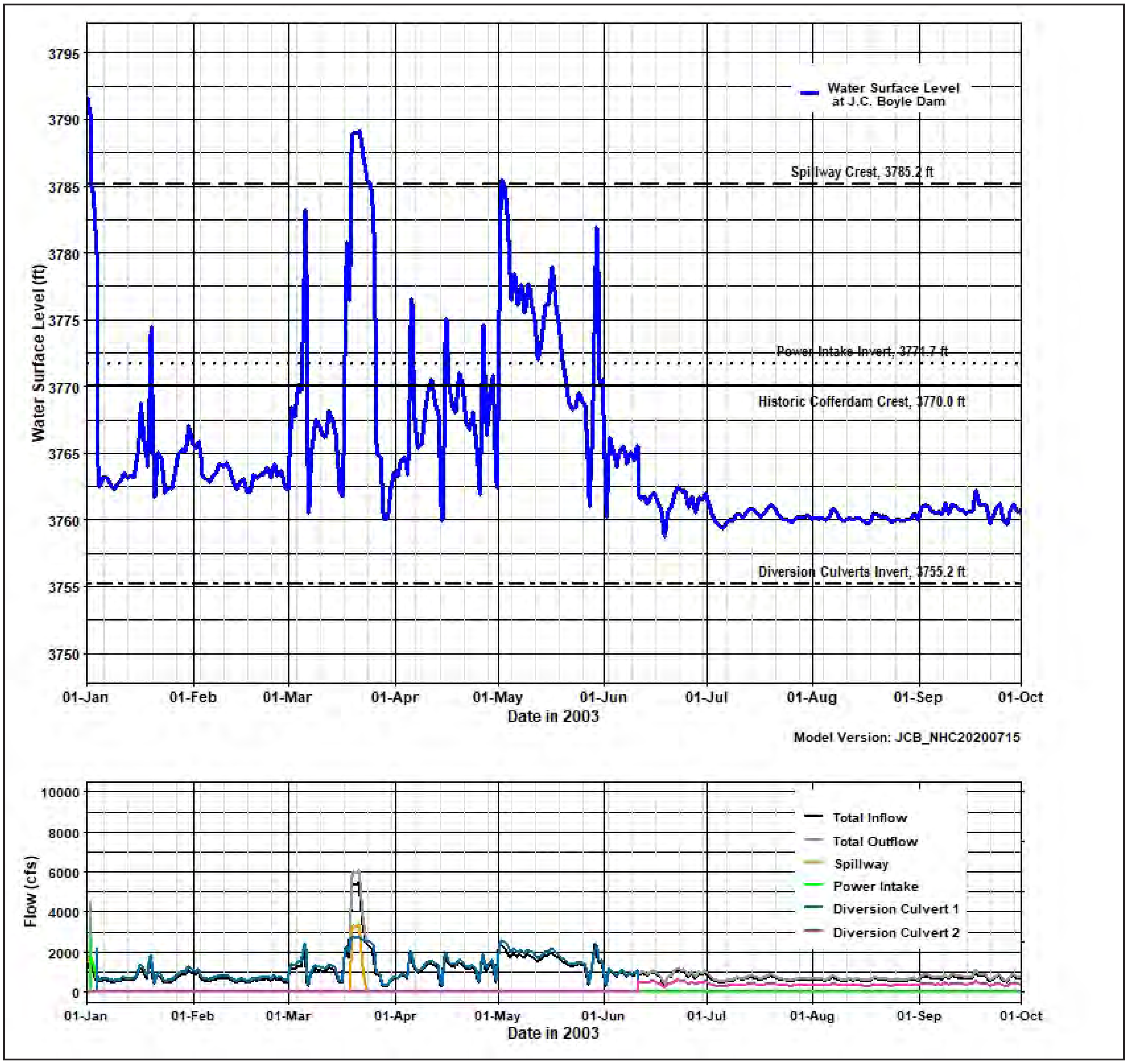

Figure 23 - J.C. Boyle Facility Simulated Drawdown - Year 2003

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J.C. BOYLE FACILITY SIMULATED DRAWDOWN

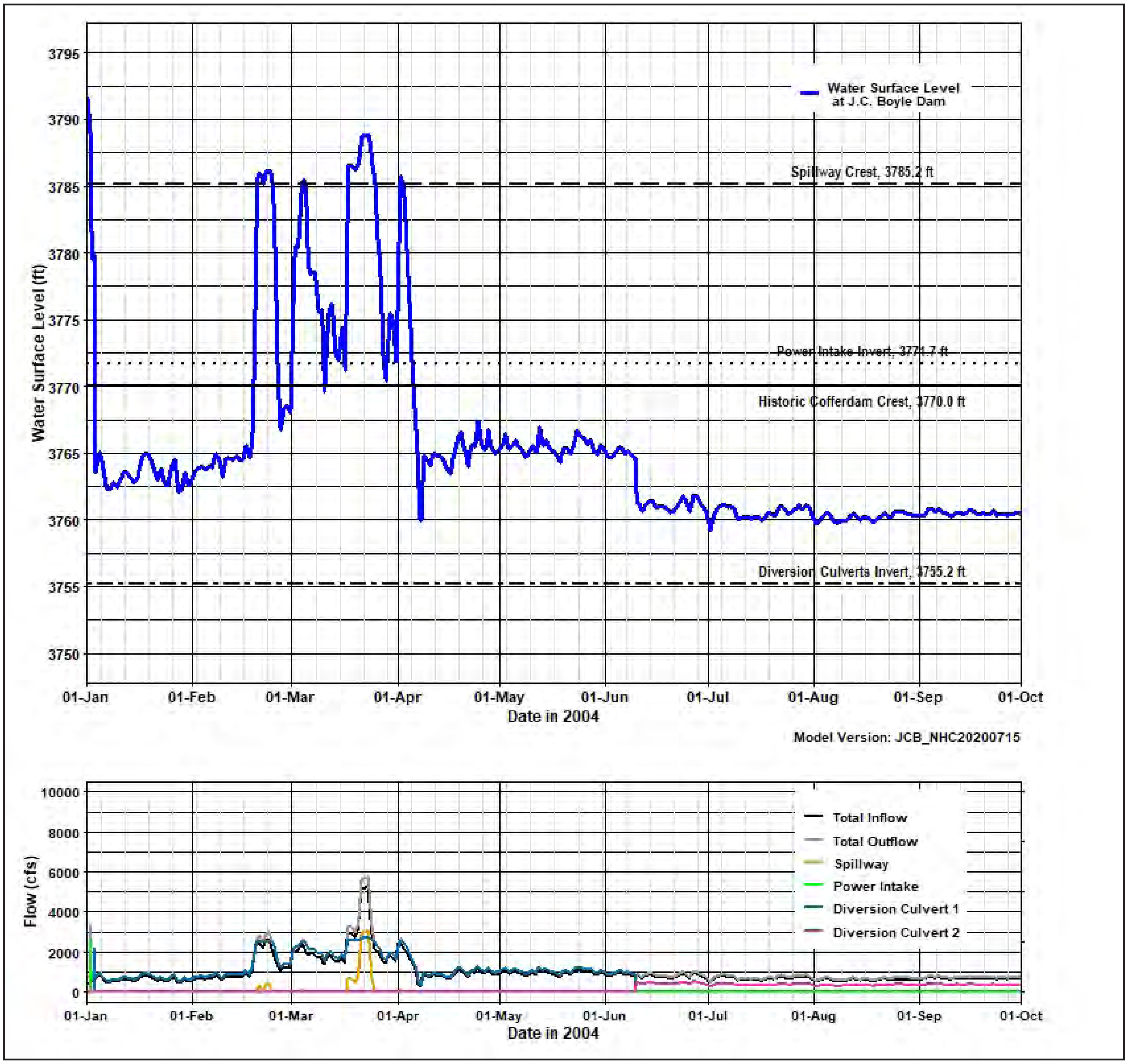

Figure 24 - J.C. Boyle Facility Simulated Drawdown - Year 2004

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J.C. BOYLE FACILITY SIMULATED DRAWDOWN

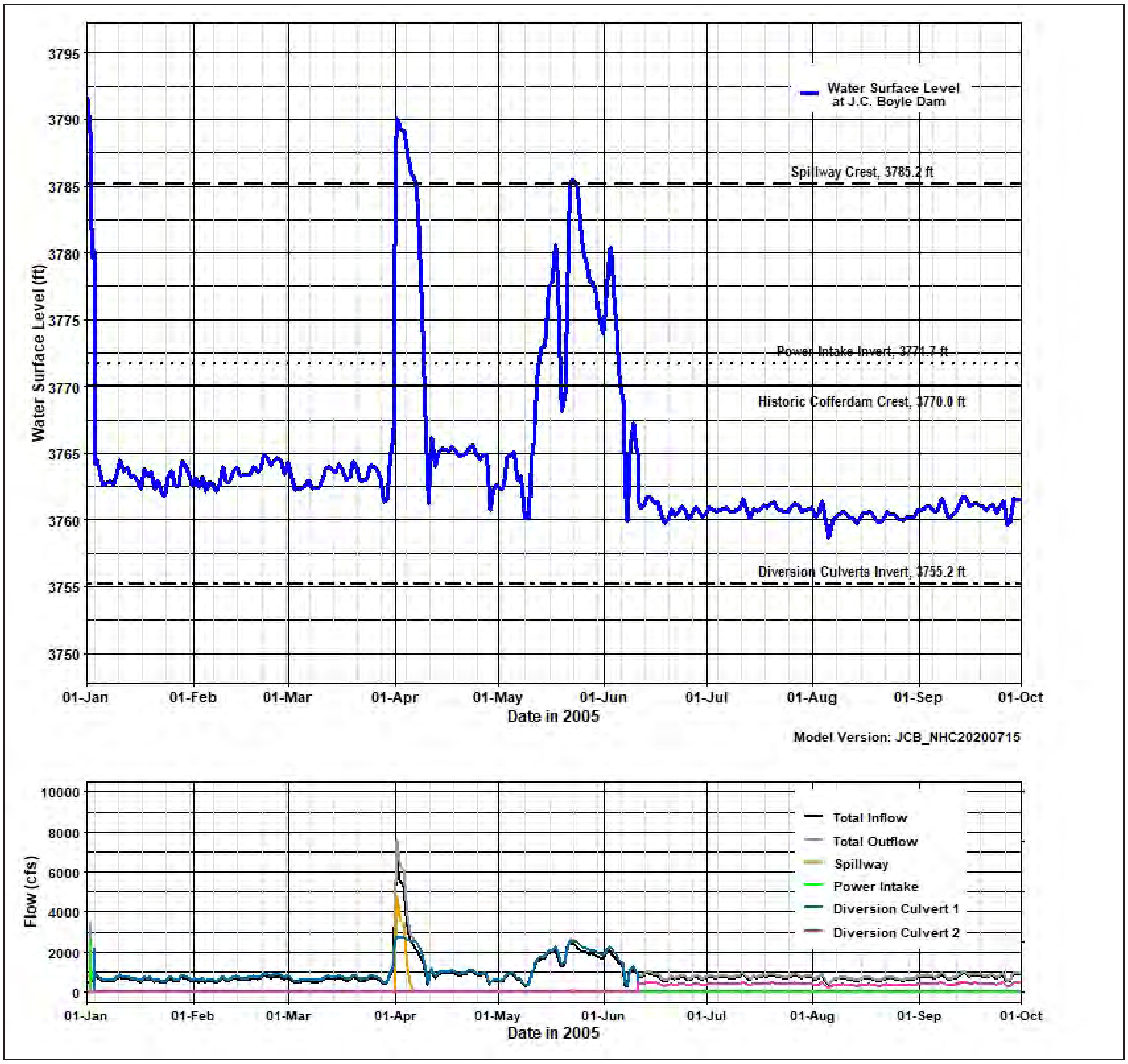

Figure 25 - J.C. Boyle Facility Simulated Drawdown - Year 2005

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J.C. BOYLE FACILITY SIMULATED DRAWDOWN

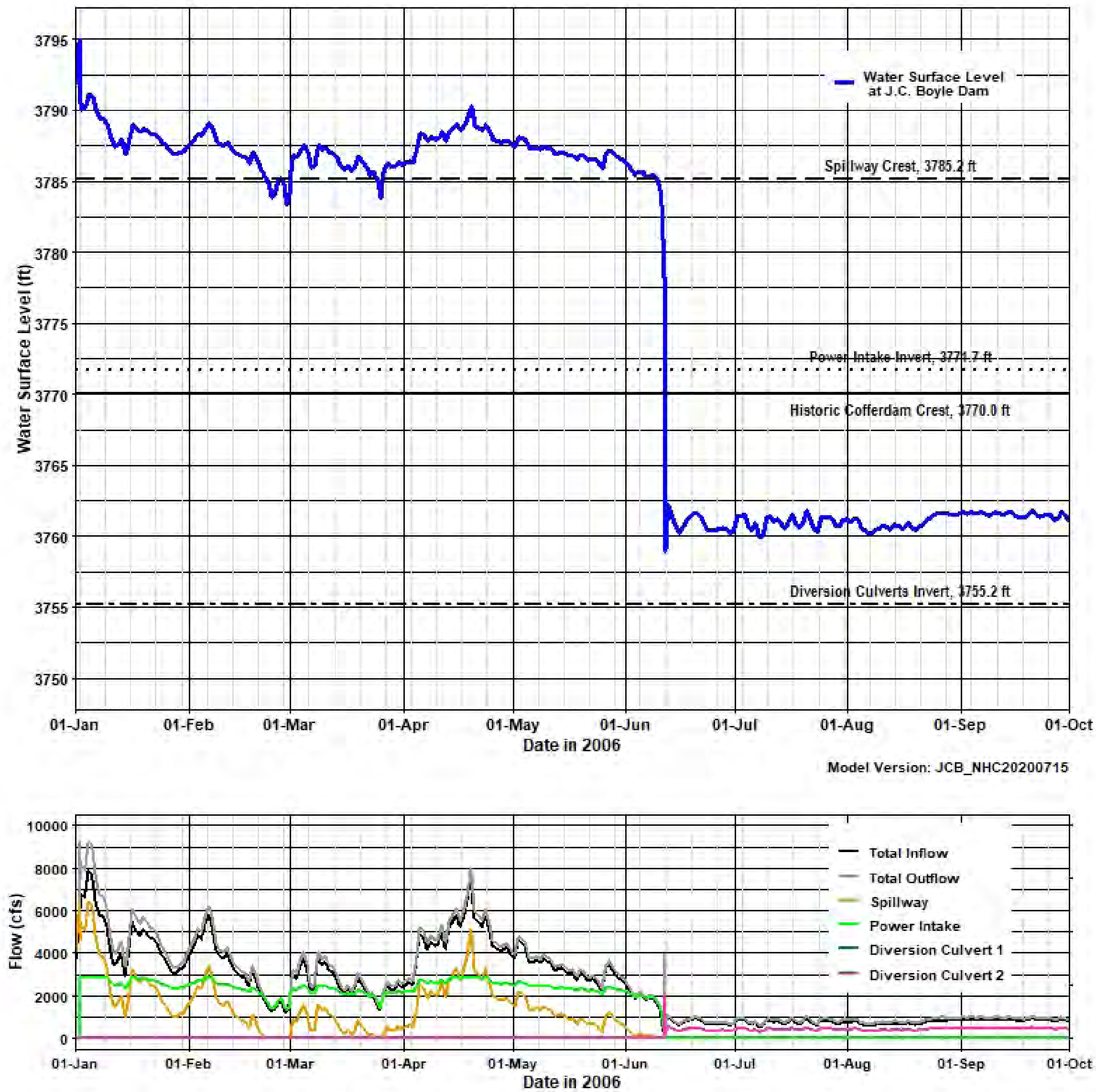

Figure 26 - J.C. Boyle Facility Simulated Drawdown - Year 2006

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J.C. BOYLE FACILITY SIMULATED DRAWDOWN

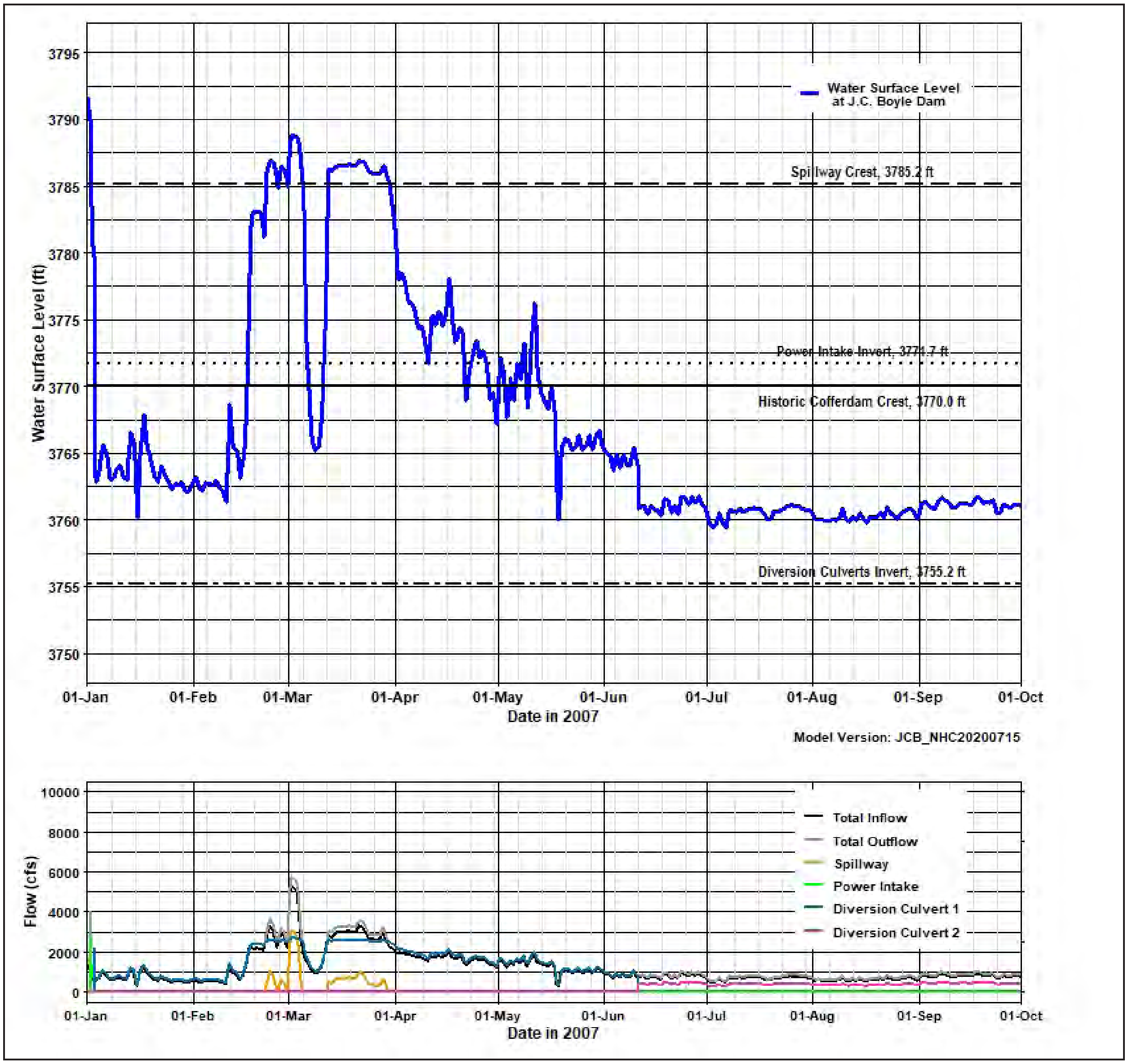

Figure 27 - J.C. Boyle Facility Simulated Drawdown - Year 2007

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J.C. BOYLE FACILITY SIMULATED DRAWDOWN

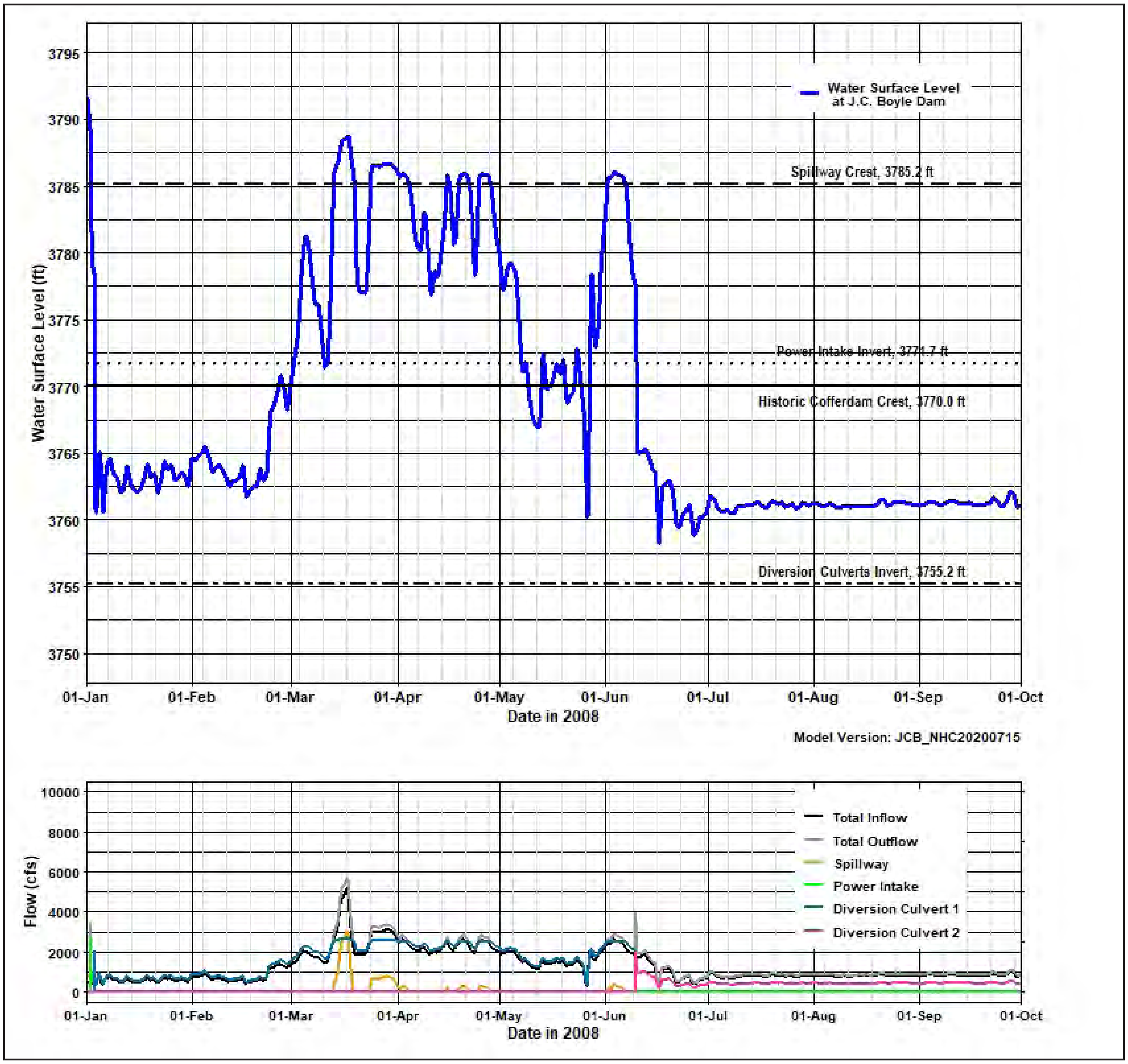

Figure 28 - J.C. Boyle Facility Simulated Drawdown - Year 2008

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J.C. BOYLE FACILITY SIMULATED DRAWDOWN

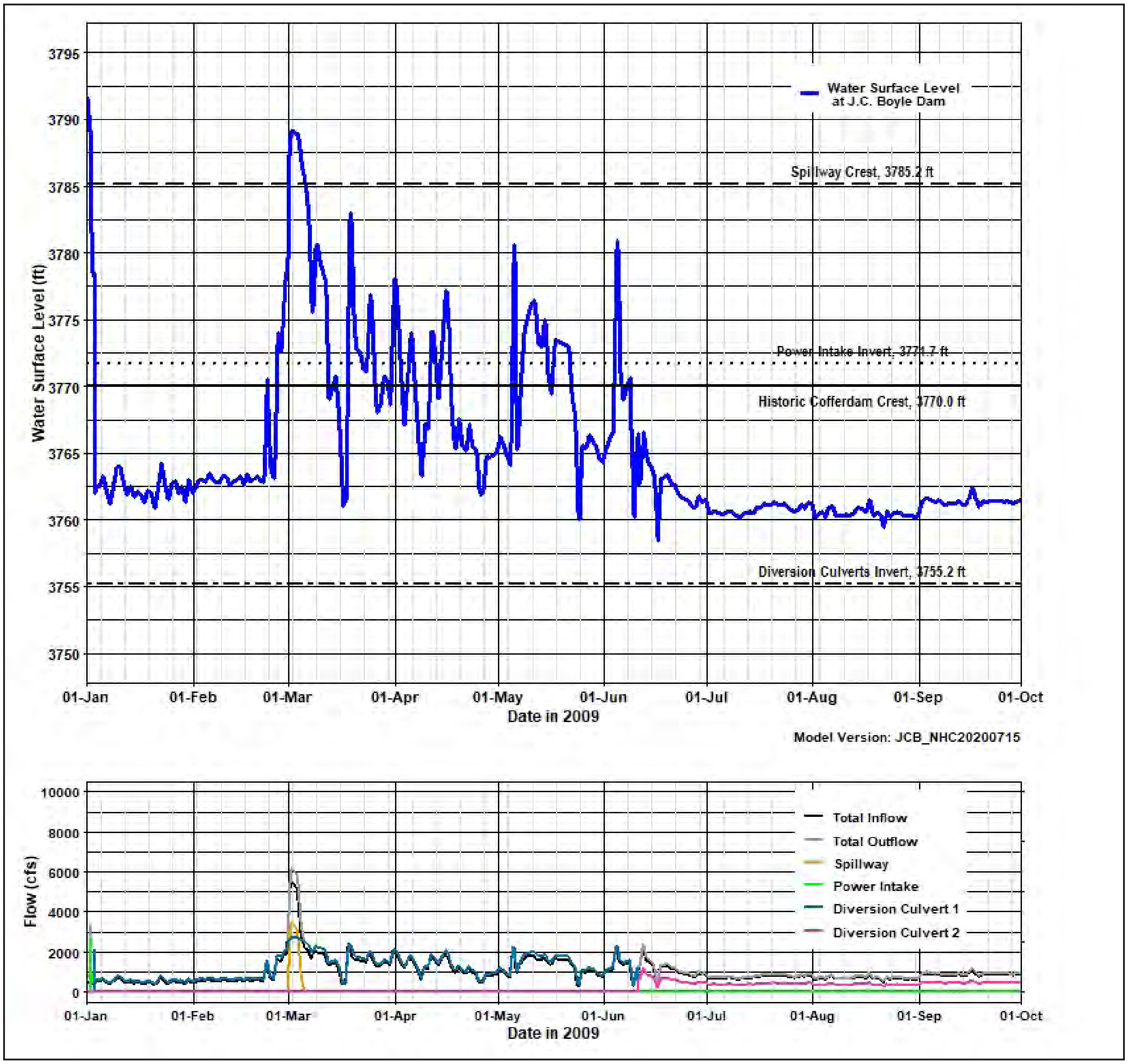

Figure 29 - J.C. Boyle Facility Simulated Drawdown - Year 2009

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J.C. BOYLE FACILITY SIMULATED DRAWDOWN

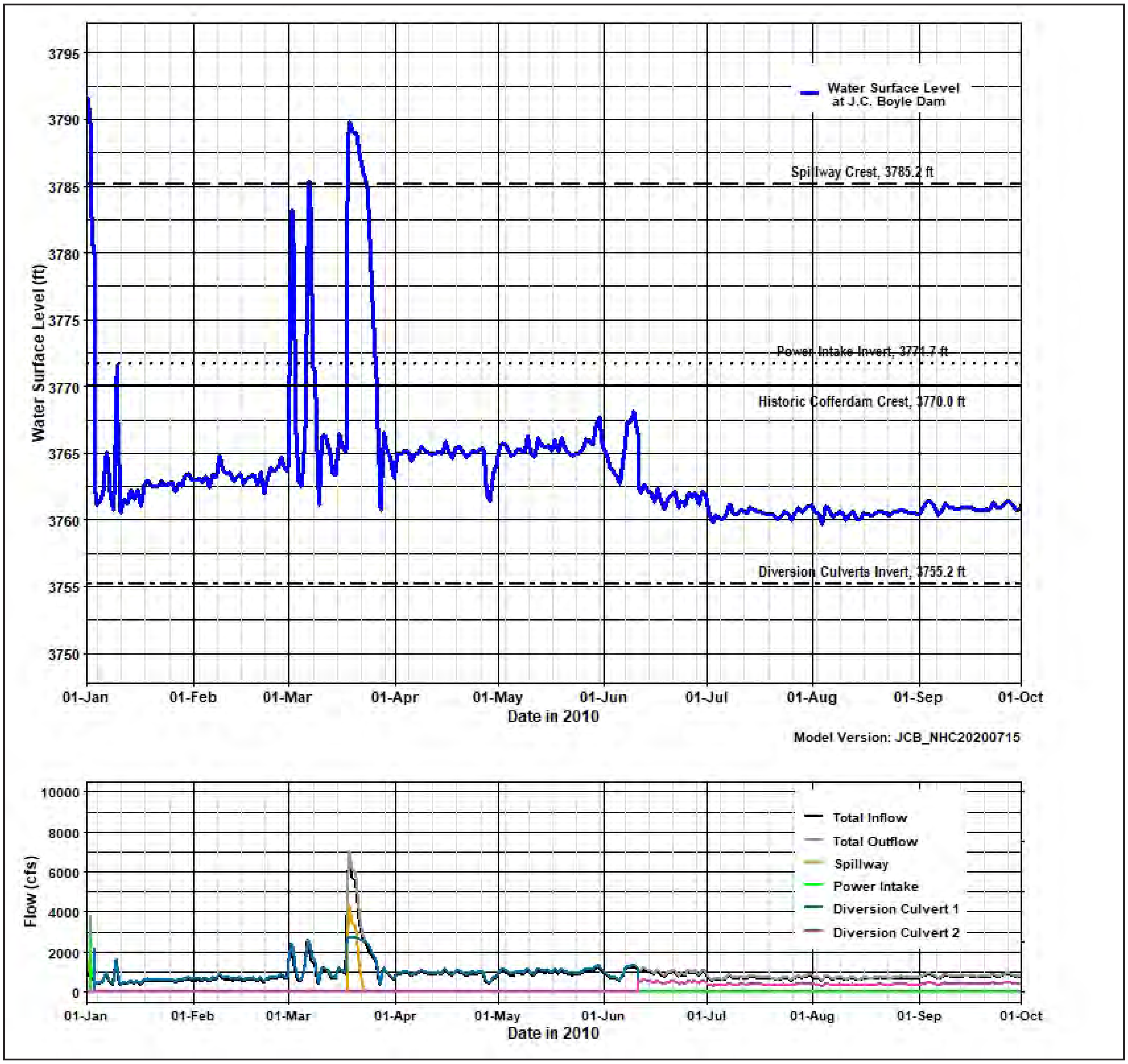

Figure 30 - J.C. Boyle Facility Simulated Drawdown - Year 2010

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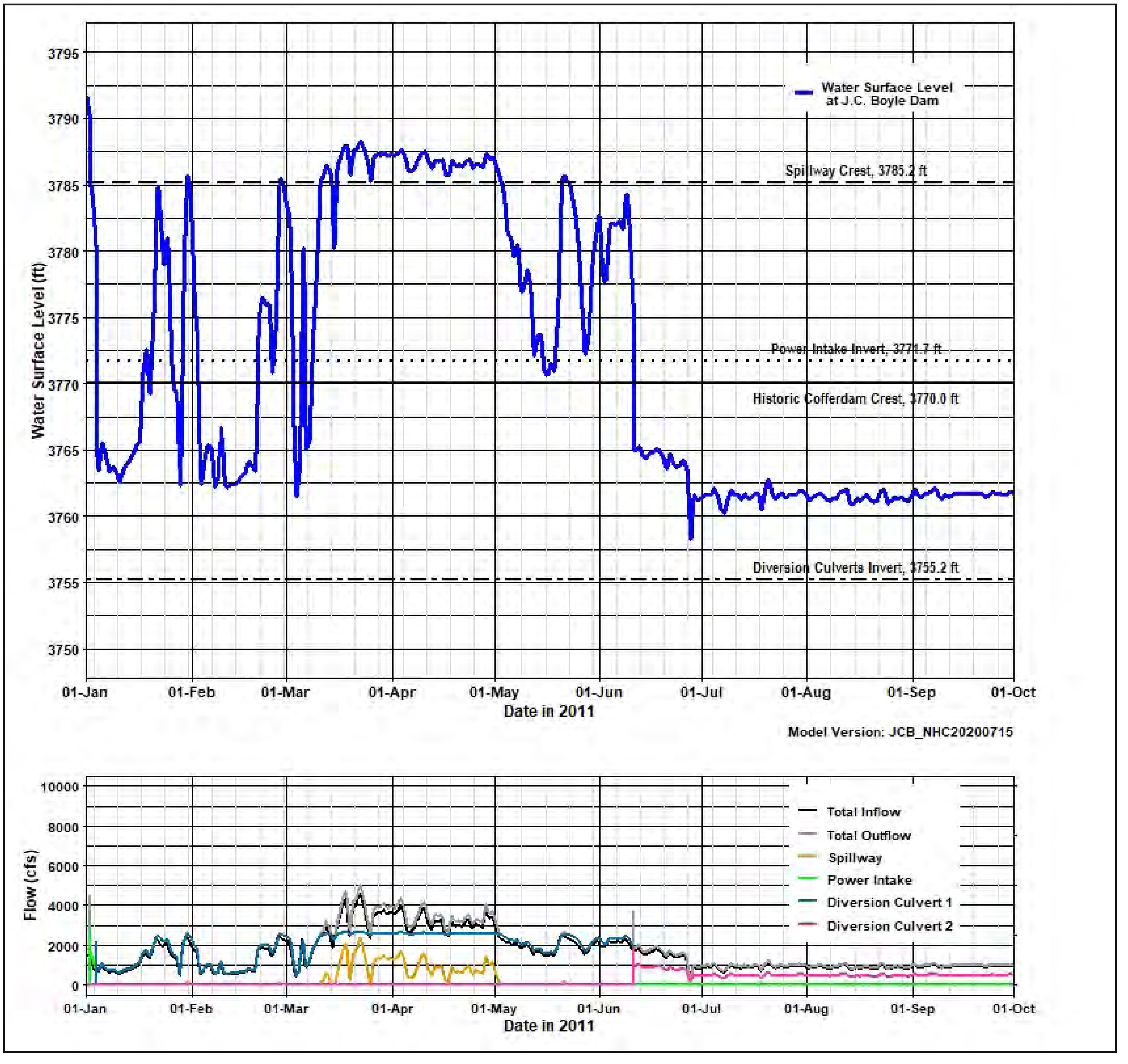

Figure 31 - J.C. Boyle Facility Simulated Drawdown - Year 2011

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J.C. BOYLE FACILITY SIMULATED DRAWDOWN

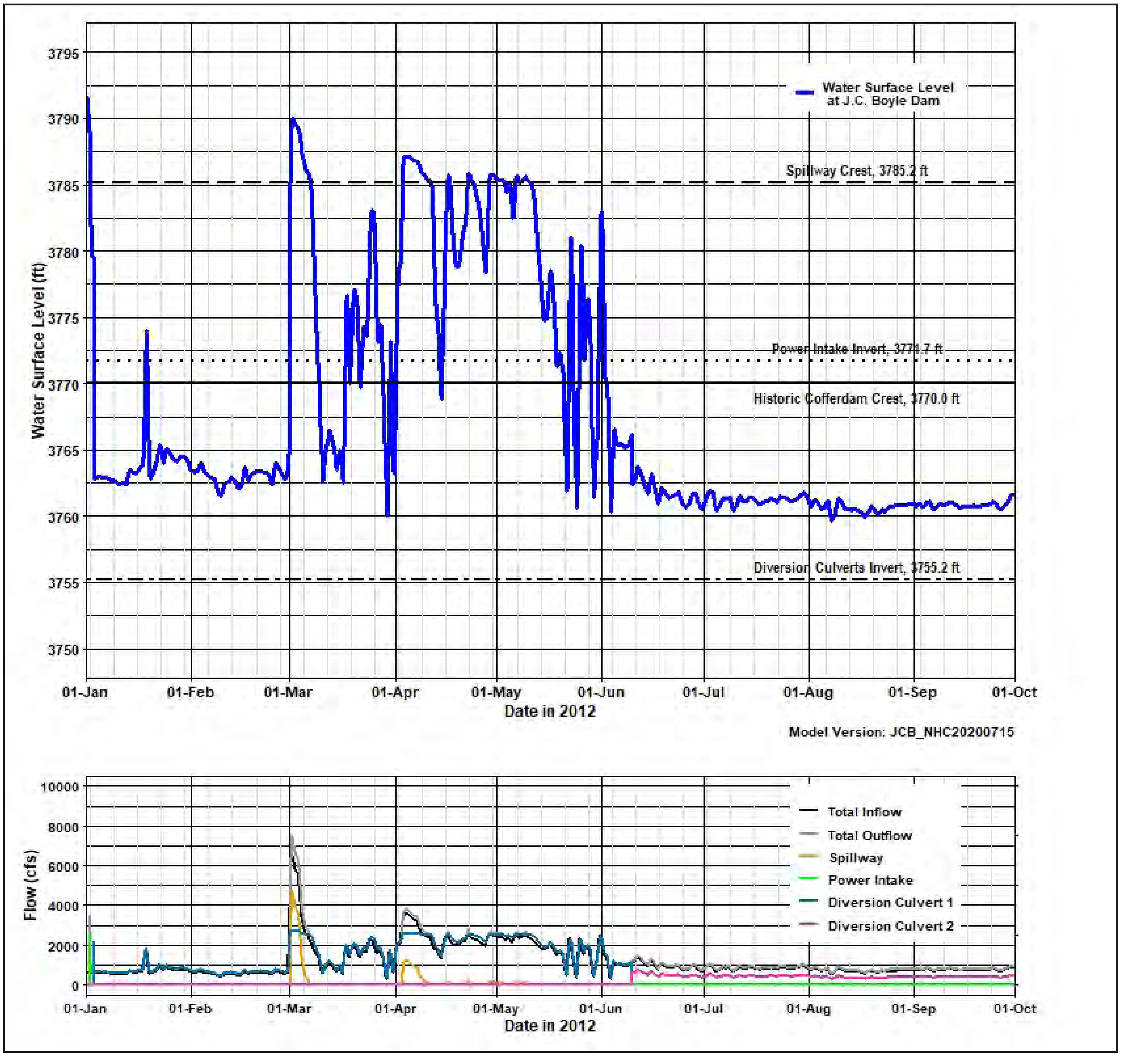

Figure 32 - J.C. Boyle Facility Simulated Drawdown - Year 2012

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J.C. BOYLE FACILITY SIMULATED DRAWDOWN

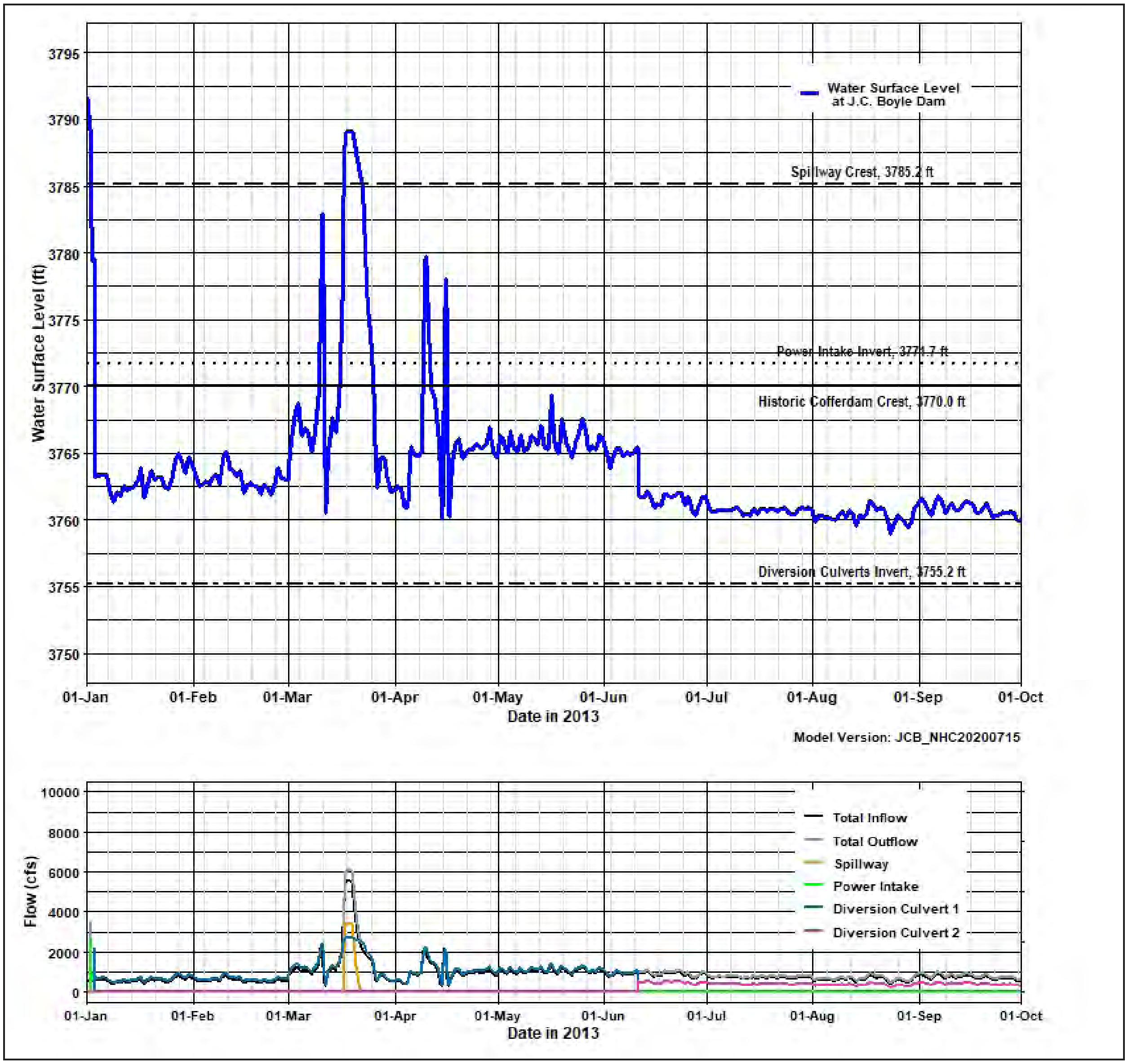

Figure 33 - J.C. Boyle Facility Simulated Drawdown - Year 2013

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J.C. BOYLE FACILITY SIMULATED DRAWDOWN

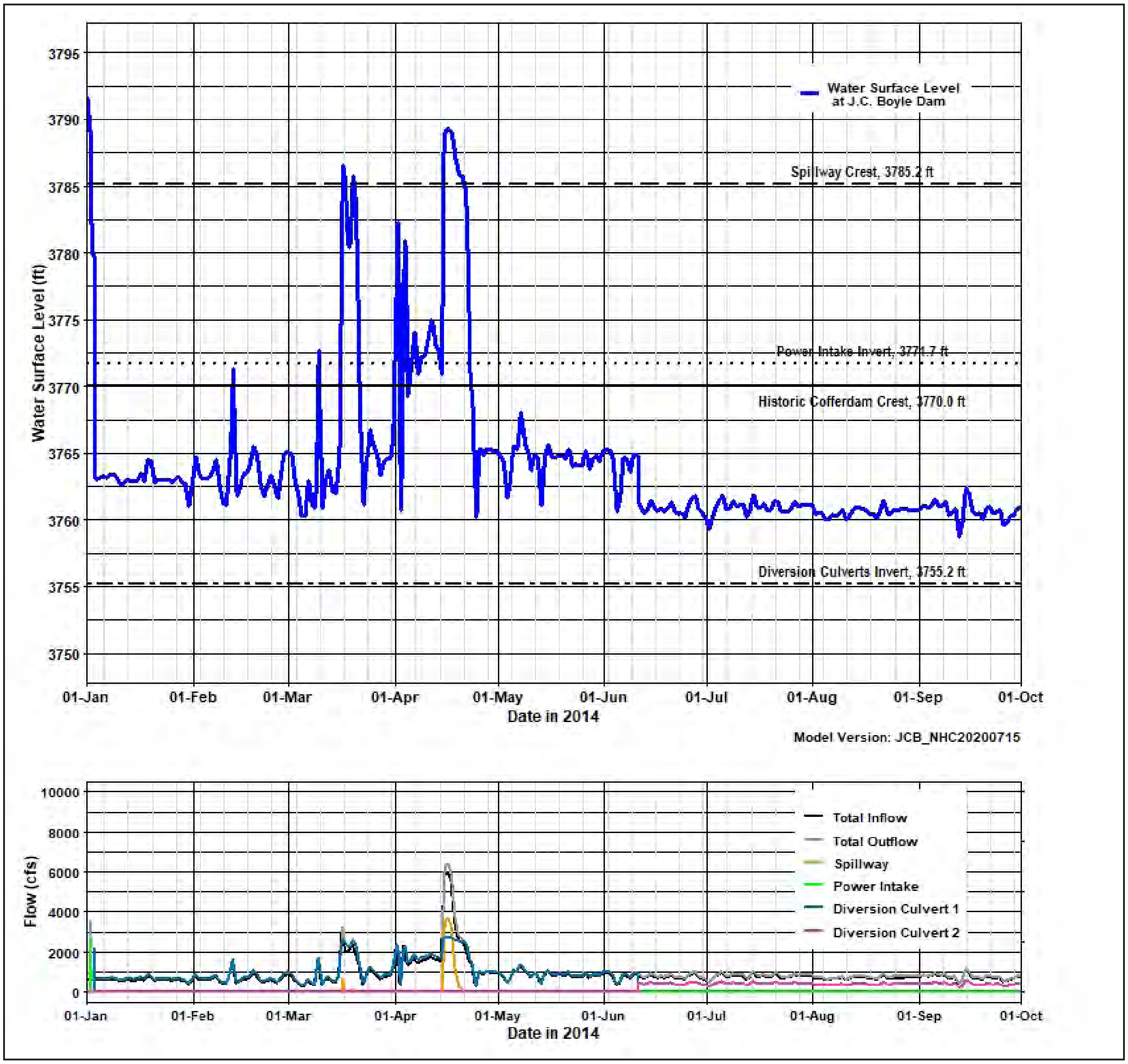

Figure 34 - J.C. Boyle Facility Simulated Drawdown - Year 2014

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J.C. BOYLE FACILITY SIMULATED DRAWDOWN

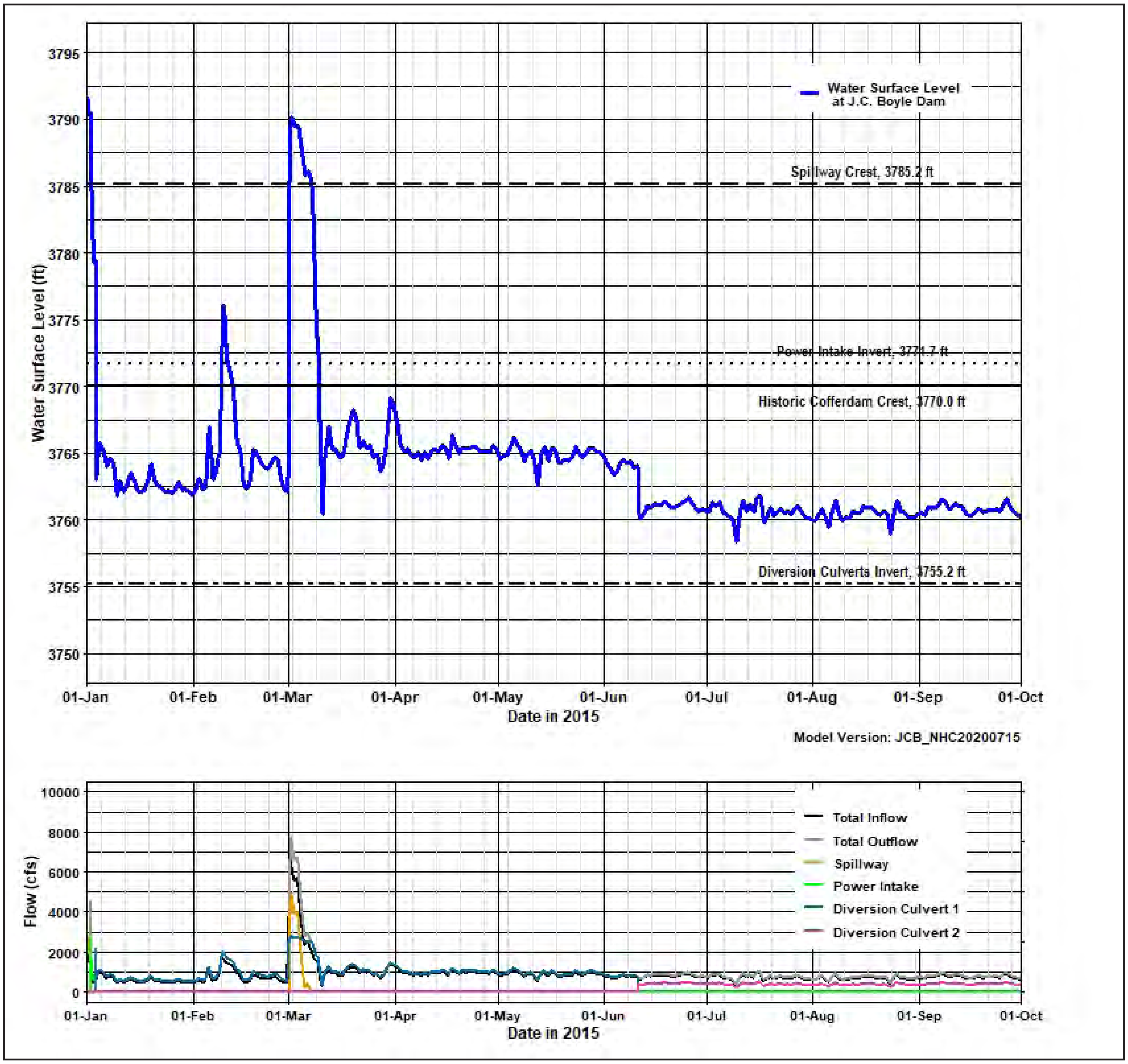

Figure 35 - J.C. Boyle Facility Simulated Drawdown - Year 2015

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J.C. BOYLE FACILITY SIMULATED DRAWDOWN

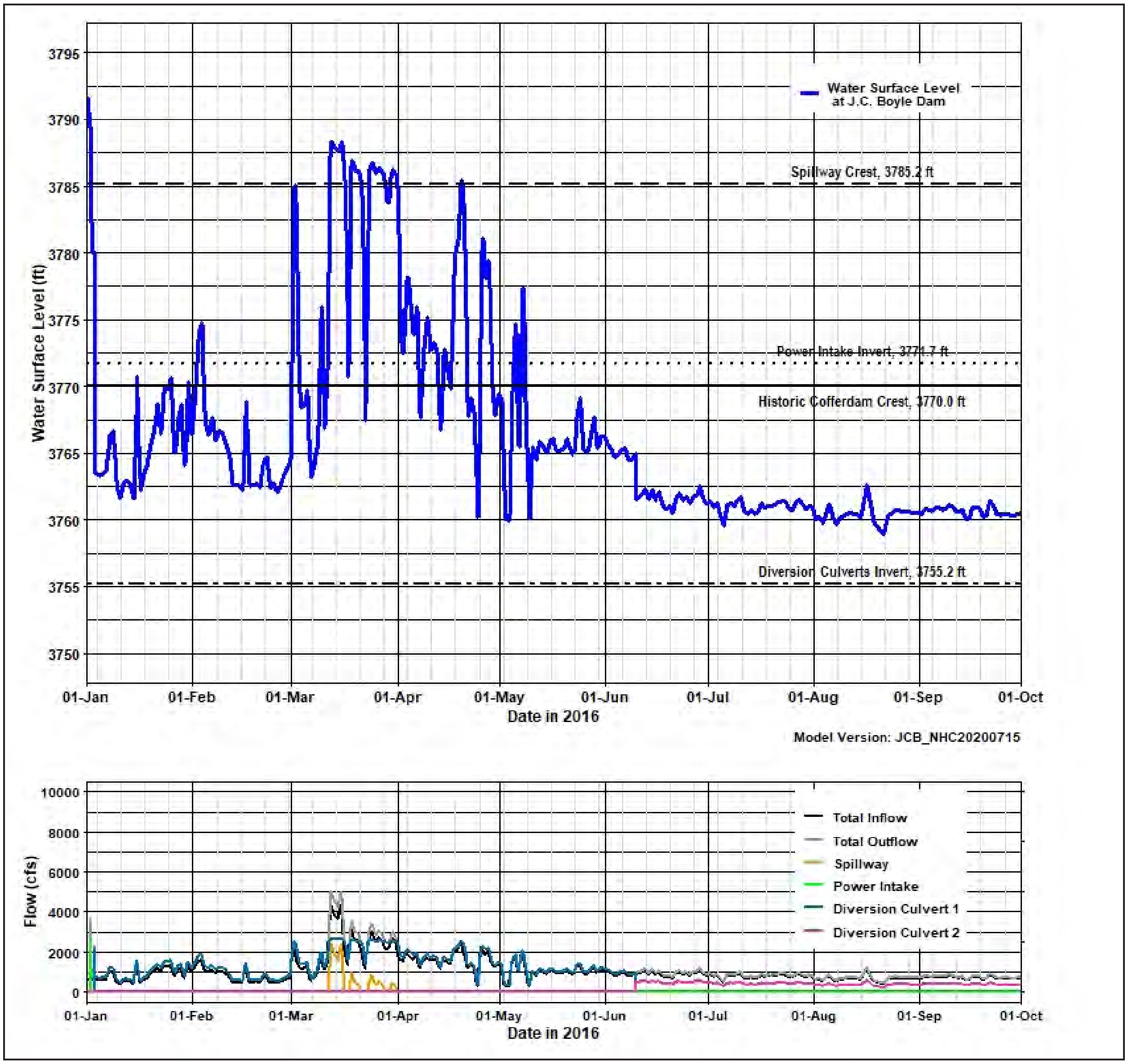


Figure 36 - J.C. Boyle Facility Simulated Drawdown - Year 2016

July 29, 2020

nhc
northwest hydraulic consultants

NHC Ref. No. 2004947.11.5

21 September 2020

Knight Piésold | KRRP Project Office
4650 Business Centre Drive
Fairfield, California, USA, 94534

Attention: **Norm Bishop**

Re: **CFD Modeling of J.C. Boyle – 100% Design**

DRAWDOWN OPERATIONS – 100% DESIGN

Knight Piésold Ltd. (KP) proposed rules for outlet operations during drawdown of the J.C. Boyle Reservoir (KP, 2020a). NHC conducted computational fluid dynamics (CFD) model simulations to develop rating curves to be applied to the drawdown modeling for the following conditions:

- Spillways and power intake are open;
- Spillways and Diversion Culvert #1 are open and power intake is closed; and,
- Spillways, Diversion Culvert #1 and Diversion Culvert #2 are open and power intake is closed.

COMPUTATIONAL FLUID DYNAMICS MODELING

NHC conducted CFD modeling of the J.C. Boyle Dam outlet structures to verify spillway and culvert capacities at various reservoir water surface elevations (RWS). This information will be used to assist the design work for drawdown and be used to adjust rules for the J.C. Boyle one-dimensional drawdown modeling. The following conditions were simulated to determine capacities at the various stages KP identified for drawdown (Table 1):

1. Reservoir at normal operating water level of RWS = El. 3,796.7 ft; all three spillway gates fully open and power intake (invert El. 3,771.7 ft) diverting 2,800 cfs. This simulation is intended to verify the spillway can pass 15,400 cfs. Note, the power intake may be operated to divert up to 2,850 cfs; however, this is considered within the tolerance of the model and model results.
2. RWS = El. 3,780.6 ft, which is below the crest of the spillway (El. 3,785.2 ft). Power intake assumed closed. This simulation is intended to verify the capacity of Culvert #1.
3. RWS = El. 3,766.0 ft. This simulation is intended to verify the combined flow capacity of Culvert #1 and Culvert #2.

Table 1. Simulated Conditions

Simulation	RWS (ft., NAVD88)	Spillway	Intake	Culvert 1	Culvert 2
1	3,796.7	Fully open	2,800 cfs	Closed	Closed
2	3,780.6	0 cfs	0 cfs	Open	Closed
3	3,766.0	0 cfs	0 cfs	Open	Open

GEOMETRY AND ROUGHNESS

The model terrain includes topo-bathymetric data (GMA, 2018) of the J.C. Boyle Reservoir and outlet channel. Also included are the spillway, outlet culverts, and intake structure per the 1956 plans (elevations in project datum, Ref. Drawings G-7585, G-8215, G- 8337, AA-78084-A, AA-78085A, AA-78087A, AA-78114A). All project elevations were converted from the historical project datum to NAVD88 using a +3.7 ft conversion. All elevations mentioned in this memo are in NAVD88 project datum.

The J.C. Boyle ogee spillway comprises three 36-ft wide bays separated by 4.5-ft wide piers. The crest of the spillway is at El. 3,785.2 ft and the top of the spillway deck at El. 3,803.7 ft. The normal operating level is RWS 3,796.7 ft. Below the central and right (north) bays are located the two historical diversion culverts that were used during construction and are currently plugged. The culverts are 9.5 ft wide and 10 ft high with invert inlet elevations at El. 3,755.2 ft. The intake tower to the 14-ft diameter steel pipe is located on the left (south) side. The pipe invert inlet elevation is El. 3,771.7 ft. Spillway gates were not incorporated in the CFD geometry. For the first simulation (Table 1), the two culverts remained plugged. For second simulation, the plug in Culvert # 1 was removed; while for the remaining simulations, the plugs from both culverts were removed.

The CFD model uses roughness height to evaluate friction losses. The roughness heights for the ground, spillway, and power intake are assumed to be 1 ft, 0.002 ft, and 0.001 ft, respectively. Model sensitivity to roughness height was not tested, as flow upstream is controlled by the spillway's weir and culvert orifice, with friction losses playing a minor role. Downstream from the dam, the river channel goes over a very steep reach and tailwater levels and roughness do not play a major influence on spillway capacity.

MESH INDEPENDENCE

The 3D mesh was developed to balance model accuracy and computation time. Decreasing the mesh cell size increases model accuracy and computation time, though there is a mesh resolution for which additional refinement does not yield significant changes in the solution. Table 2 summarizes the results of spillway capacity sensitivity to mesh cell size. The solution using the 0.25-ft cell size at the spillway is used to determine the spillway discharge coefficient, and the 1-ft cell size around the spillway is considered optimal for further simulations. The 3D mesh used for subsequent simulations includes 8-ft elements through the reservoir with a 1-ft refinement region around the spillway.

Table 2. Mesh Independence Comparison - Spillway Capacity

Cell Size (ft)		# cells	CPU	Discharge	Relative
Dam	Spillway	(million)	Time (h)	(cfs)	difference
1.00	1.00	7.8	6	18,284	-
1.00	0.50	12.5	16	17,455	-4.5%
1.00	0.25	15.5	6	17,848	2.1%

SIMULATION 1 RESULTS (SPILLWAY OPEN)

Since spillway rating curves were not available, this simulation was intended to verify the spillway capacity. The reservoir was set at normal operating water level of El. 3,796.7 ft, with the all three spillway gates removed and the power intake diverting 2,800 cfs. Under these conditions, average spillway capacity is approximately 17,850 cfs.

Figure 1 below illustrates velocity variations over the spillway. The spillway outlet channel is relatively steep with the spillway high above the ground, so backwater or a submerged tailwater condition are not expected. The water jet reaches a maximum velocity of approximately 50 ft/s at the toe of the spillway.

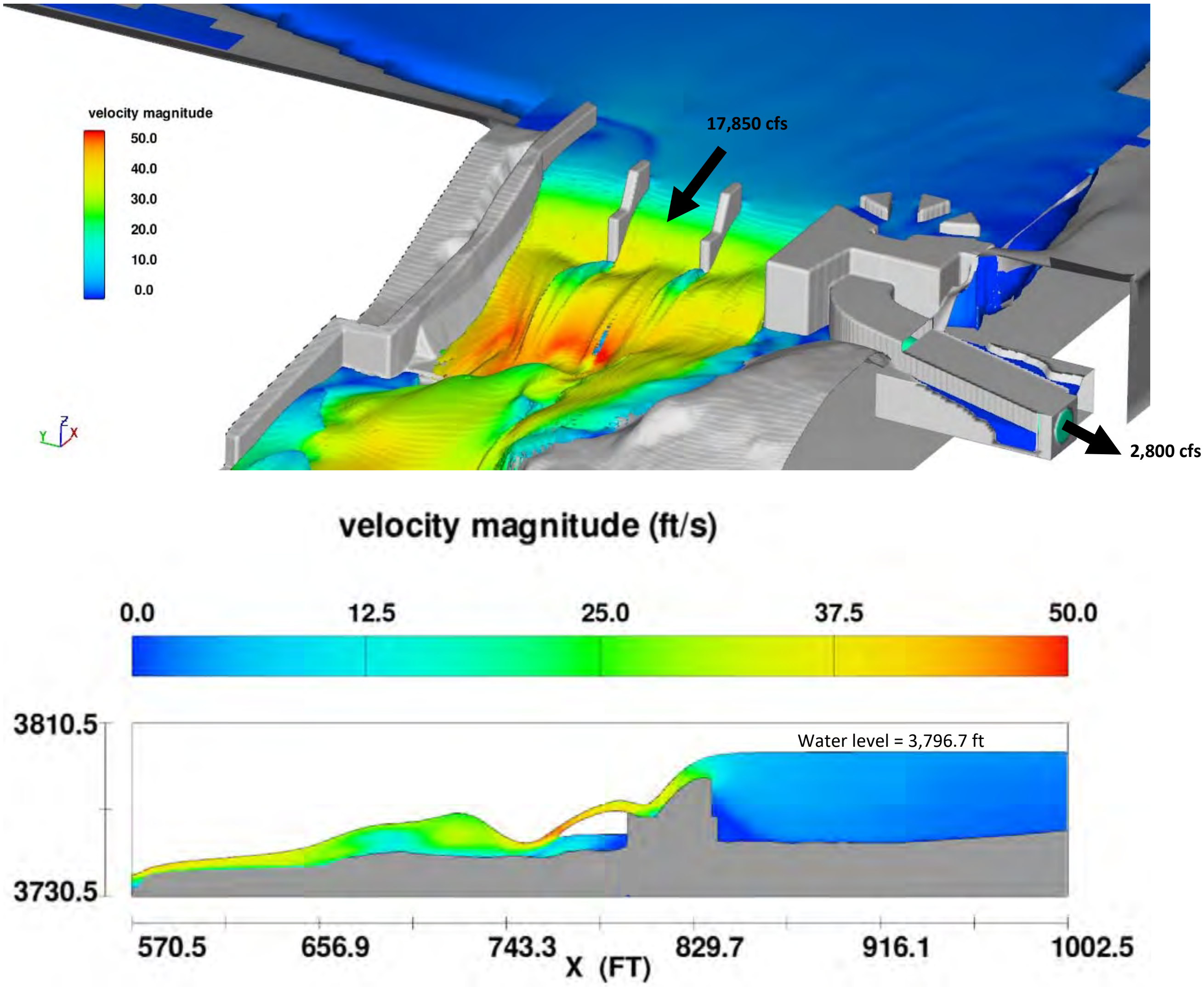
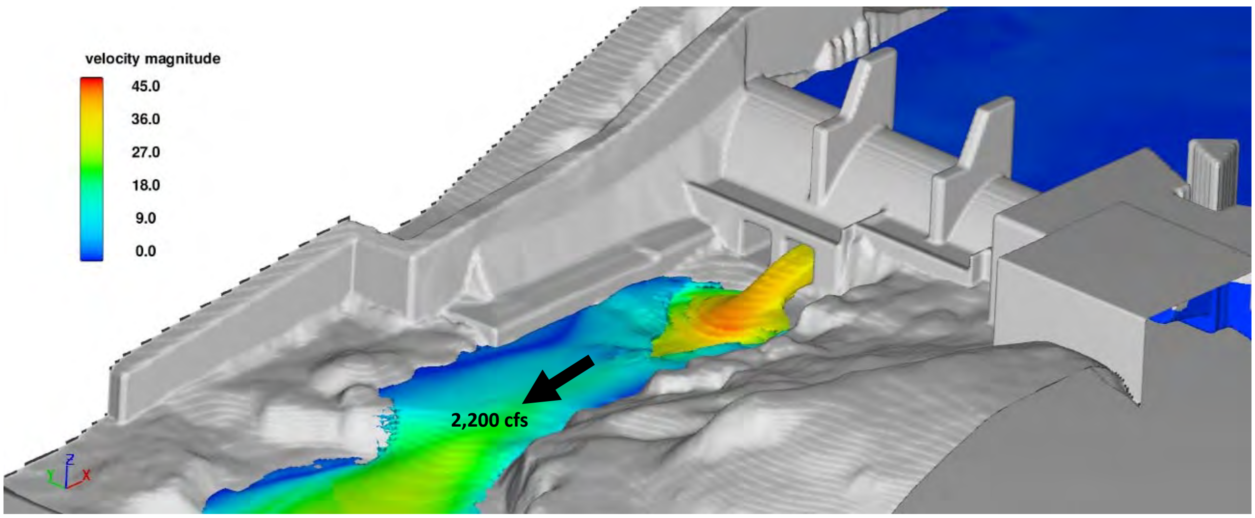


Figure 1. Results for Simulation 1 – Spillway with reservoir water level El. 3,796.7 ft.

SIMULATION 2 RESULTS (CULVERT #1 OPEN)

This simulation was conducted to provide a datapoint to verify the left (southernmost) culvert capacity (Culvert #1). The reservoir is operating at El. 3,780.6 ft (4.6 ft below the crest of the spillway) with no intake diversion (i.e. power tunnel discharge of 0 cfs). The culvert opening is assumed per the design drawings (Ref. Drawing AA-78085A, AA-78087A), rectangular with 9.5-ft span and 10-ft rise. Model results for this condition show that the culvert is not flowing full and there is no tailwater submerging the outlet (note that tailwater effects are not expected for total river flows up to at least 9,500 cfs); therefore, at this water level, the culvert capacity is inlet-controlled and flow capacity is approximately 2,200 cfs, with an orifice discharge coefficient of approximately 0.64, which is reasonable for a square edge orifice. In this scenario, flow reaches a maximum velocity of approximately 45 ft/s at the culvert outlet. Figure 2 below illustrates velocity variations at the culvert outlet.



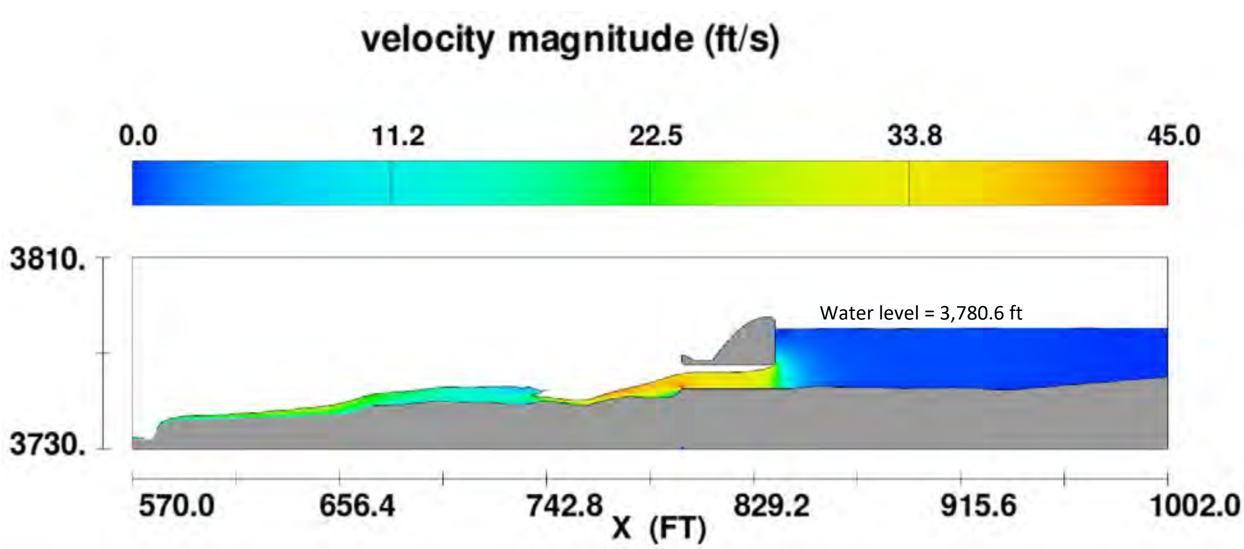


Figure 2. Results for Simulation 2 – Culvert # 1 open with reservoir water level El. 3,780.6 ft.

SIMULATION 3 RESULTS (TWO CULVERTS OPEN)

For both Simulations 1 and 2, the water surface in the reservoir remains relatively flat and it is easy to determine a unique RWS value. However, for Simulation 3, there was a strong gradient in the water surface as the water concentrated and accelerated along the steep historically excavated approach channel upstream of the diversion culverts. Because of this, two CFD simulations with two different waters levels were conducted for Simulation 3.

The first, Simulation 3A, provided capacity verification of both culverts when the reservoir was operating at El. 3,766 ft with no power intake diversion. The selected water level of El. 3,766 ft corresponded to a location in the approach channel approximately 300 feet upstream of the dam, near the historical cofferdam, where the slope of the approach channel is approximately 5 percent. When the approach channel water surface is at El. 3,766 ft, this corresponded to a water surface of approximately El. 3,761.3 ft just upstream of the dam, and a flow of approximately 900 cfs through the culverts (Figure 3A).

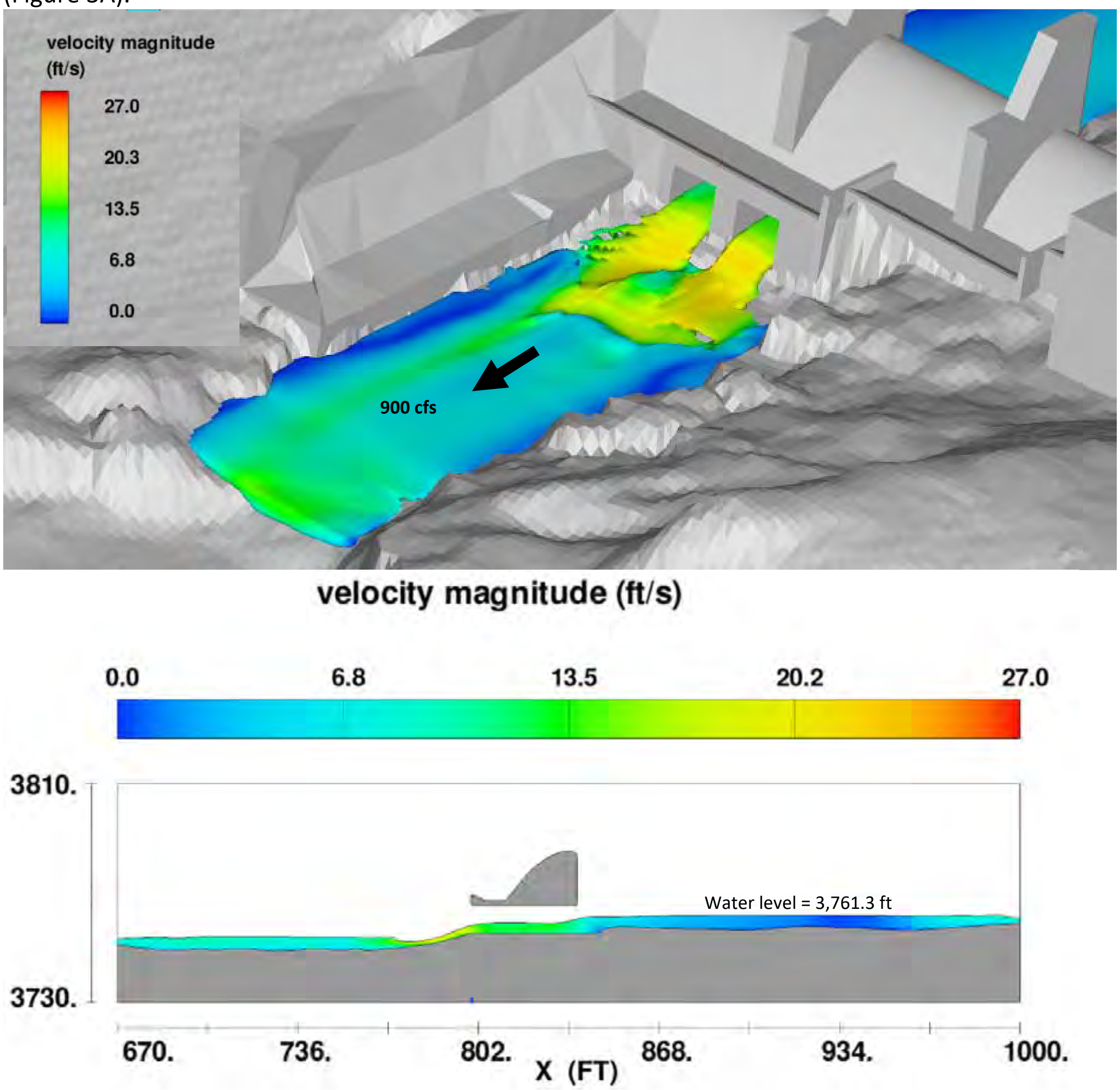
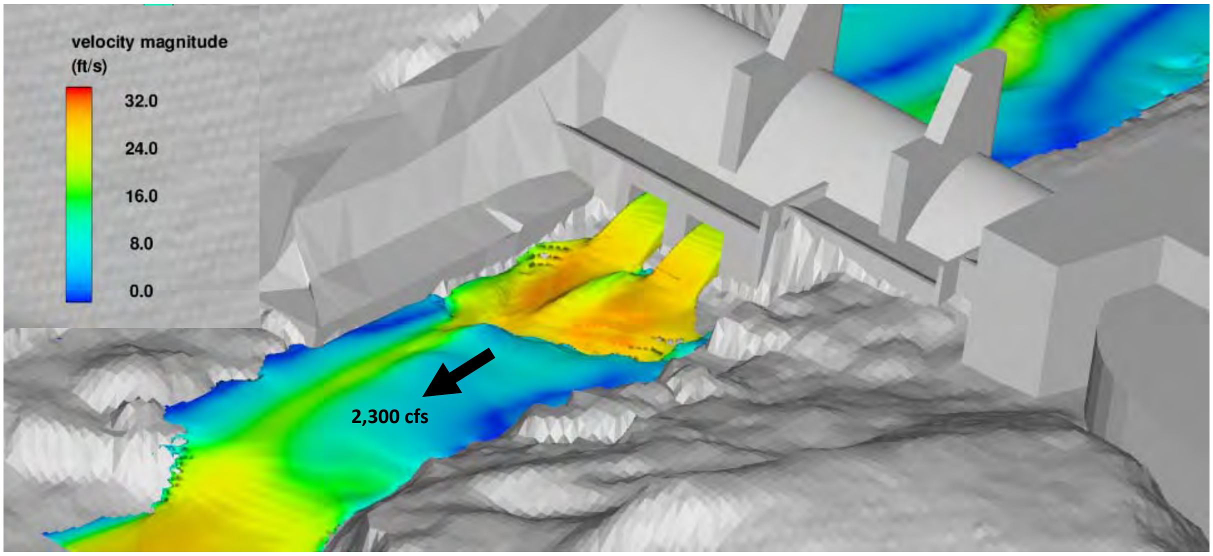


Figure 3A. Results for Simulation 3A (Two culverts - 900 cfs)

A second Simulation 3B was conducted to determine the outlet capacity when the water surface just upstream of the dam was at El. 3,766 ft. A combined flow of approximately 2,300 cfs can be conveyed by both culverts when the water surface just upstream of the dam is at El. 3,766 feet. The orifice discharge coefficients become 0.59 for Culvert # 1 and 0.62 for Culvert # 2. Figure 3B below illustrates velocity variations at the outlet of the culverts for this condition.



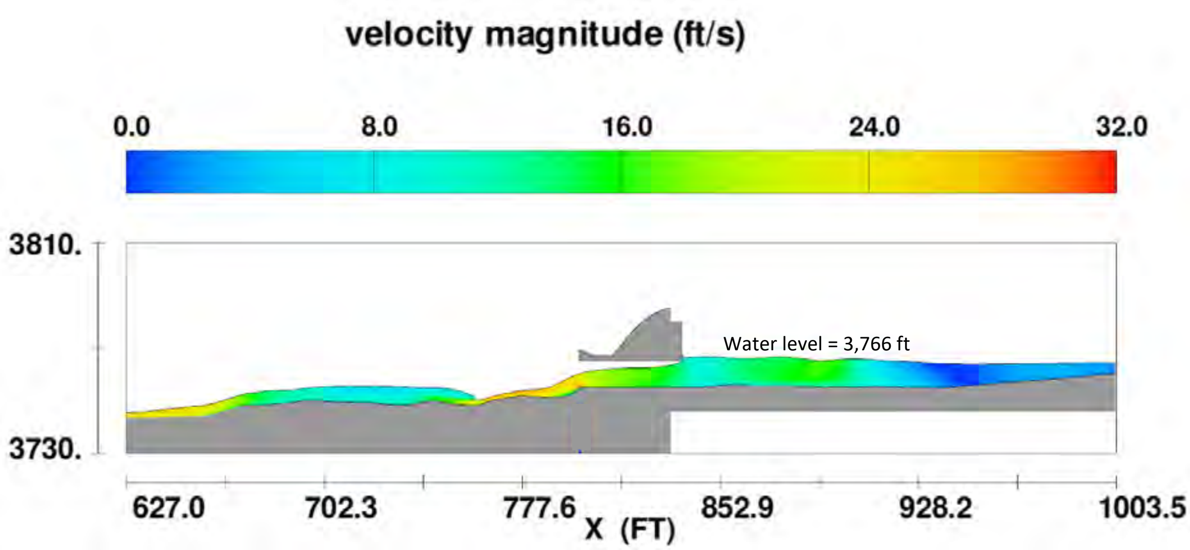


Figure 3B. Results for Simulation 3B (Two culverts - 2,300 cfs)

RATING CURVES

Figure 4 shows the discharges modeled in each CFD simulation as described above and the rating curves for various combinations of outlets. Rating curves reference a range of discharge coefficients computed from the various modelled scenarios. The spillway discharge coefficient varies from 4.0 to 4.2 $ft^{0.5}/s$, and culvert orifice discharge coefficients vary from 0.56 to 0.77. Appendix A provides tables of the data used to produce the curves shown in Figure 4 and for integration into the 1D drawdown modeling effort.

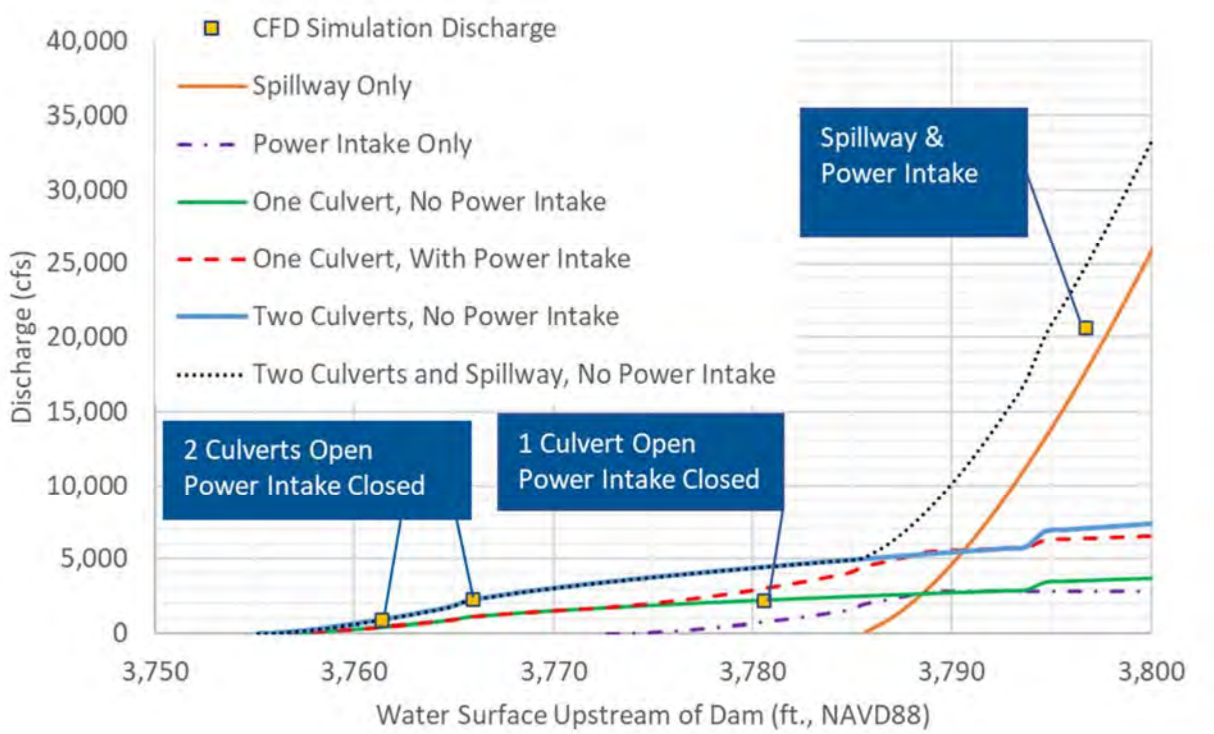


Figure 4. Rating Curves for J.C. Boyle Dam (Note: Power intake rating curve provided by KP, 2020b)

DAM EMBANKMENT WATER SURFACE ELEVATIONS

Reservoir water surface elevations at the center of the dam embankment, upstream of the dam, were requested for a range of annual probable flows when all outlets are open except the power intake. The definition of the flows was defined in the 60% Design Report (KP and RES, 2020). Computation time for this number of simulations would be significant using a CFD model, so NHC developed a HEC-RAS 2D model to assist with providing a wide range of results upstream of J.C. Boyle in a timely manner. Results include an estimate of water surface elevations at the center of the upstream embankment and shear stresses at the headpond.

The HEC-RAS 2D model uses the post-drawdown outlet rating curve for the spillway and two culverts developed from the CFD model results as a 2D flow area connection to simulate the dam outlet works. Notice that water levels at the embankment are higher than at the concrete structure (spillway and culverts), especially during low flows which generate a strong water surface gradient. For example, for a discharge of 900 cfs, the water level difference is 6 ft.

Figure 5 shows the estimated reservoir water surface elevations at the embankment. Appendix A provides tables of the data used to produce Figure 5. Figures 6 through 9 illustrate the FLOW 3D and 2D model simulations of localized drawdown at the outlet structure and resulting variability in water surface elevations between the outlet structure and the embankment.

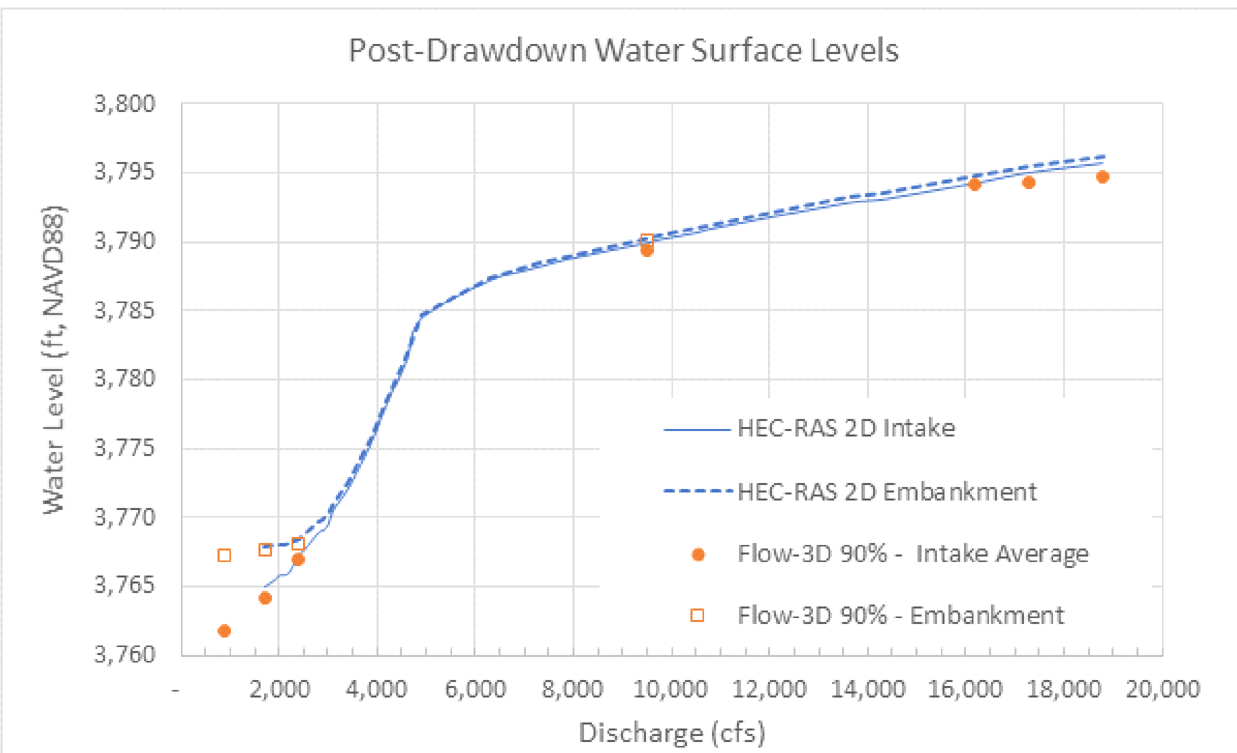


Figure 5. Reservoir Water Surface Elevations at the Intake and Embankment predicted by 2D and 3D models

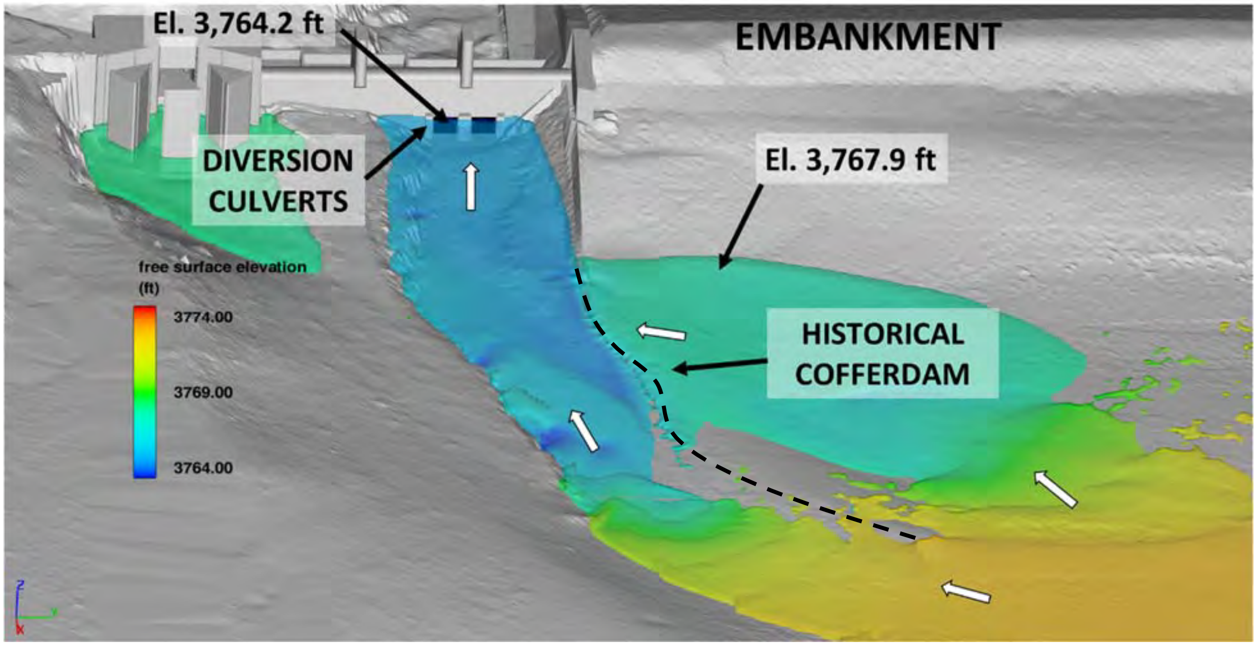


Figure 6. Reservoir Water Surface Elevations (ft) at the Embankment at 1,700 cfs (Using FLOW-3D)

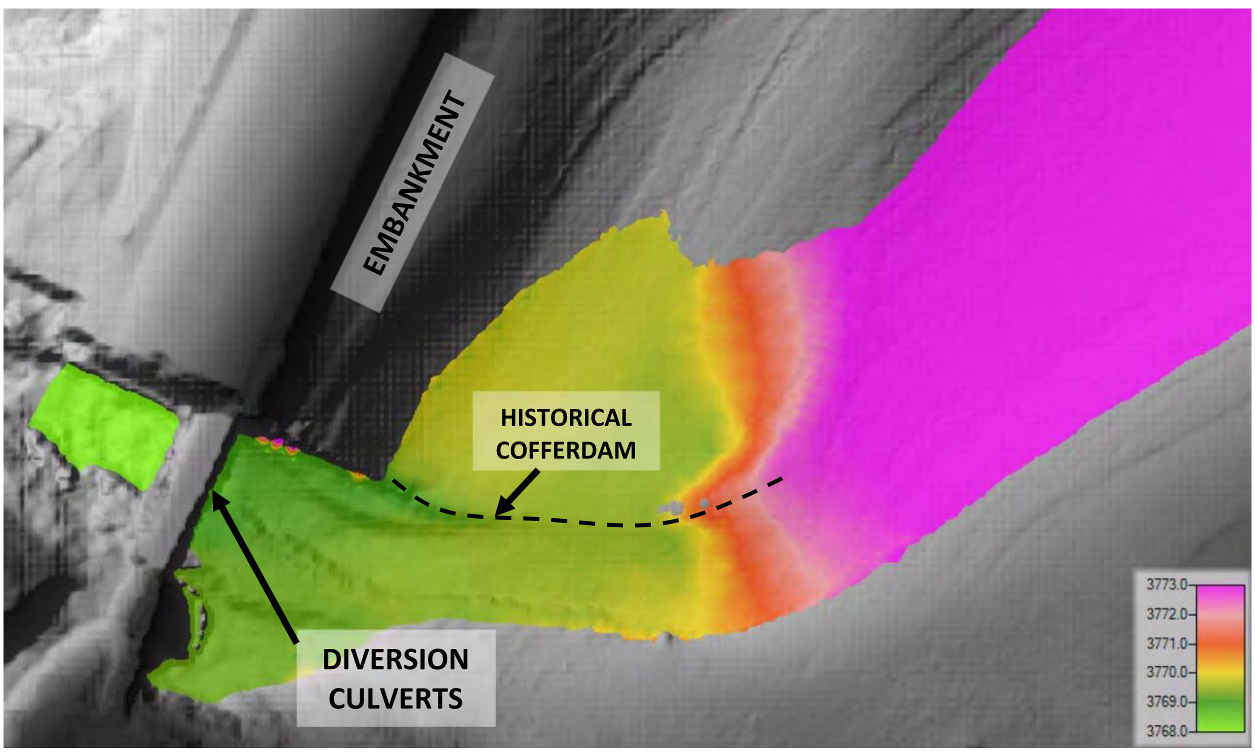


Figure 7. Reservoir Water Surface Elevations (ft) at the Embankment at 2,800 cfs

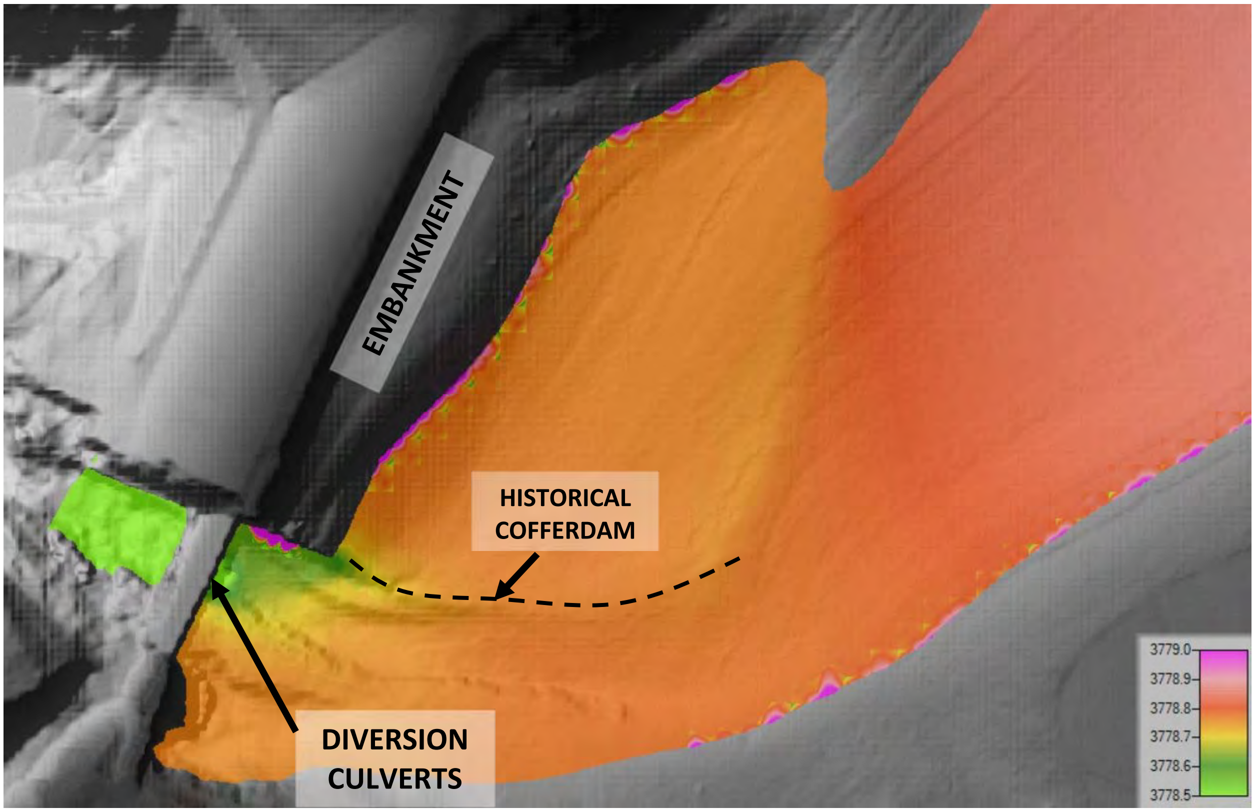


Figure 8. Reservoir Water Surface Elevations (ft) at the Embankment at 4,200 cfs

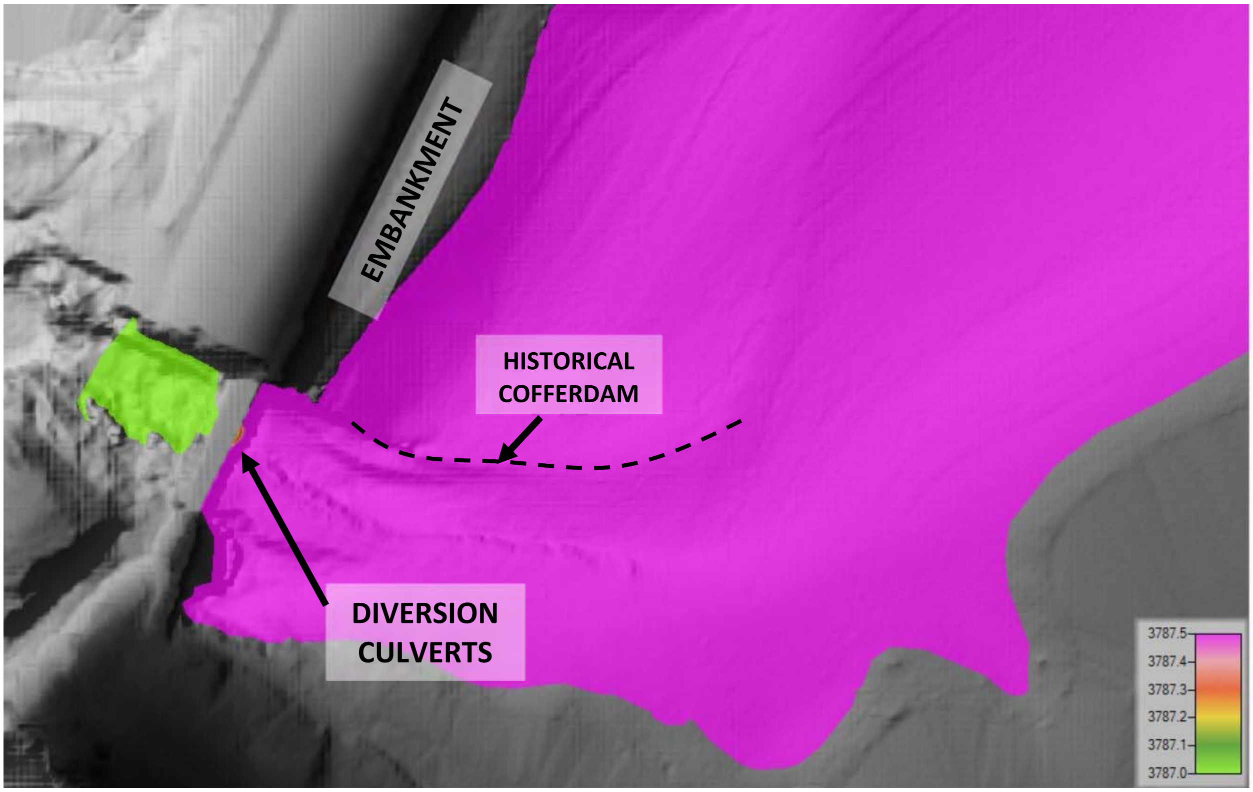


Figure 9. Reservoir Water Surface Elevations (ft) at the Embankment at 6,700 cfs

SEDIMENT MOBILITY

The potential of flow to mobilize sediment at the upstream headpond was assessed by comparing the bed shear stress of the flow predicted by FLOW-3D and HEC-RAS 2D with the critical bed shear stress shown in Table 3. The critical shear stress values presented in Table 3 approximately represent the minimum shear force applied by the flow per unit area of the bed, needed to initiate the motion of a sediment particle of a given size, which is surrounded by particles of the same size resting on a flat bed.

Table 3. Critical shear stress to initiate sediment motion on flat bed. Adapted from Julien (2002).

Sediment class name	Particle size		Critical shear stress	
	(mm)	(ft)	(Pa)	(lbf/ft^2)
Small boulder	256	0.8	223	4.7
Large cobble	128	0.4	111	2.3
Small cobble	64	0.2	53	1.1
Very coarse gravel	32	0.1	26	0.54
Coarse gravel	16	0.05	12	0.25
Medium gravel	8	0.03	5.7	0.12
Fine gravel	4	0.01	2.71	0.057
Very fine gravel	2	0.01	1.26	0.026
Very coarse sand	1	0.003	0.47	0.010

UPSTREAM RESERVOIR AND HEADPOND

Figure 10 to 13 illustrate shear stresses near the headpond during Simulations 1, 2, and 3. Table 4 below compares the shear stresses, and largest particle size anticipated to be mobilized, at the headpond during each event. During Simulations 1 and 2, velocities in the approach channel generate shear stresses capable of mobilizing medium to very coarse gravels. Once drawdown is complete and both culverts are opened, a larger flow event during post-drawdown operations could generate local scour at the headpond outlet as shear stress near the headpond is great enough to mobilize large cobbles and small boulders. The duration for which these culverts will operate at these velocities will be determined during drawdown and is dependent on reservoir water levels and inflow rates. Drawdown modeling results may be reviewed in a subsequent design phase to determine possible ranges of time that culverts and low-level outlets will be operated under abrasive conditions.

Table 4. Anticipated Sediment Mobility

Simulation	Reservoir WS (ft NAVD88)	Flow (cfs)	Shear Stress at Headpond (lbf/ft^2)	Mobilized Sediment Class	Mobilize Particle Size (in)
1	3,796.7	17,850	0.5	Very coarse gravel	1.3
2	3,780.6	2,200	0.1	Medium gravel	0.3
3B	3,766.0	2,300	1	Small Cobble	2.5
3A	3,761.3	900	3	Large Cobble	6.5

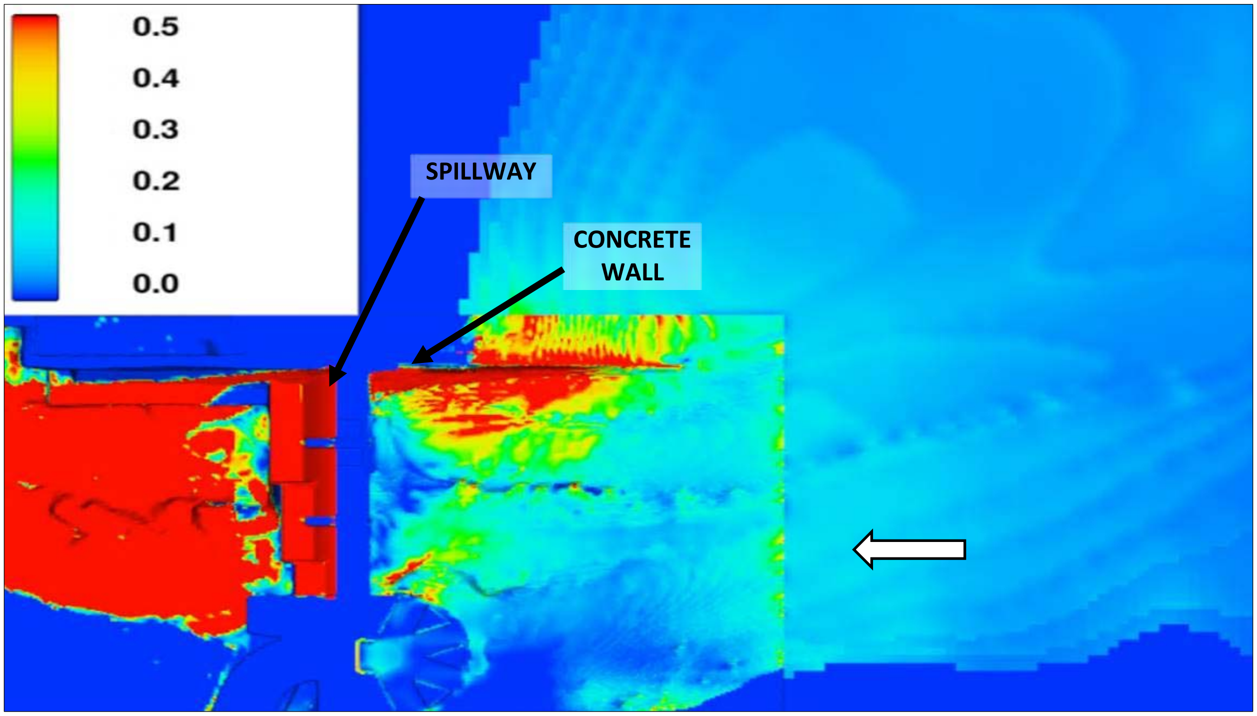


Figure 10. Simulation 1, Shear Stress Magnitude (lbf/ft^2), maximum shear stresses < 0.5 lbf/ft^2

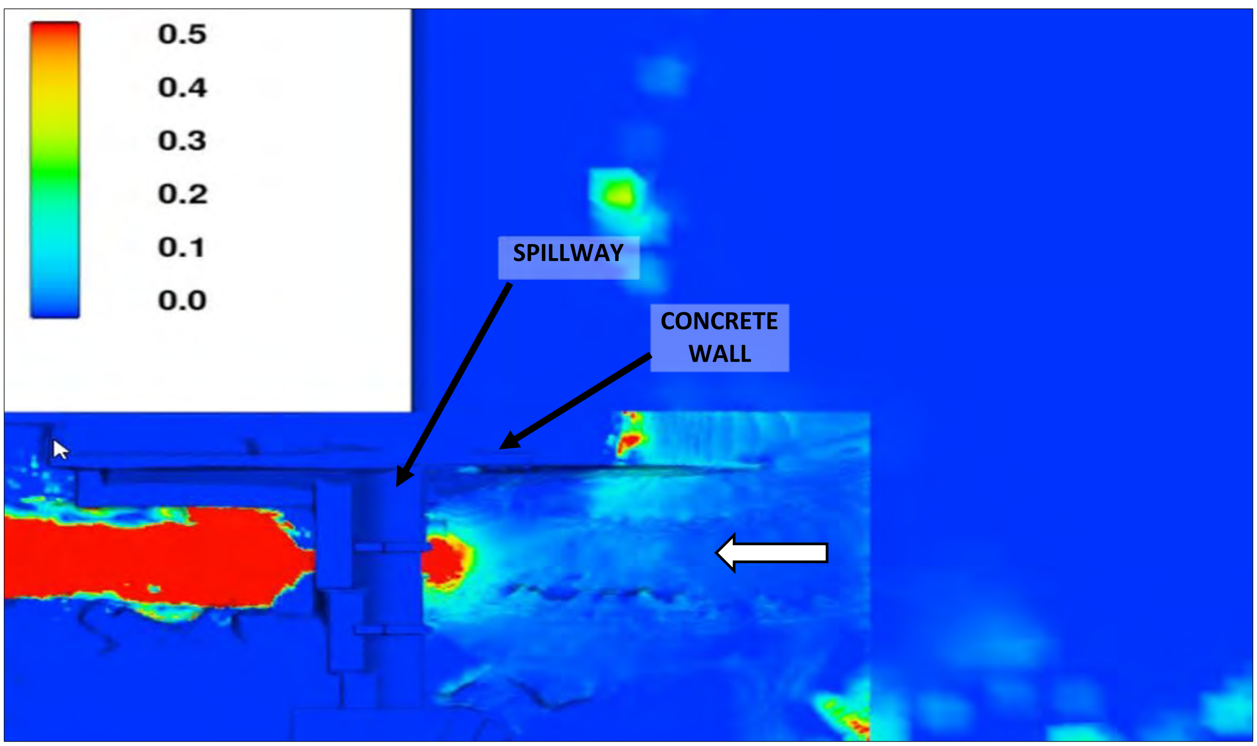


Figure 11. Simulation 2, Shear Stress Magnitude (lbf/ft^2), maximum shear stresses < 0.1 lbf/ft^2

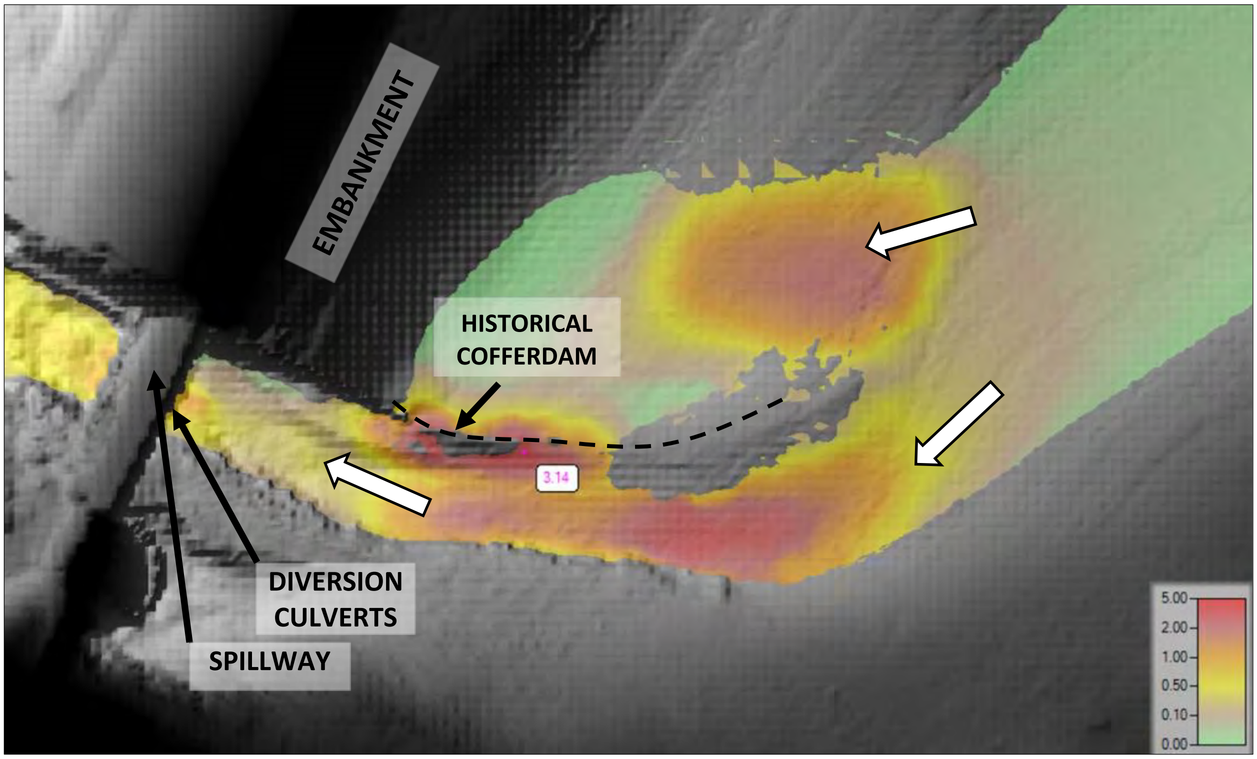


Figure 12. Simulation 3A, Shear Stress Magnitude (lbf/ft^2), Q = 900 cfs, maximum shear stresses ~ 3 lbf/ft^2

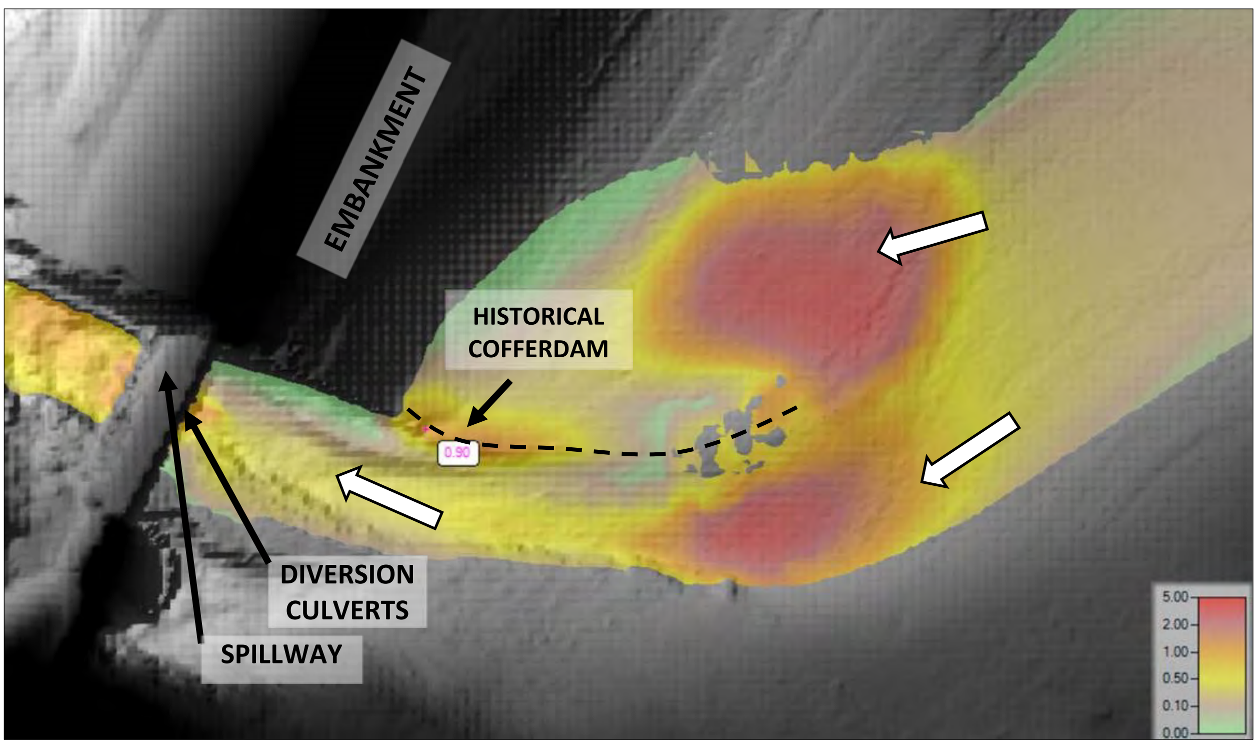


Figure 13. Simulation 3B Shear Stress Magnitude(lbf/ft^2), Q = 2,300 cfs, maximum shear stresses < 1 lbf/ft^2

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APPENDIX A

J.C. BOYLE TOTAL DISCHARGE CAPACITY AND DRAWDOWN OPERATIONS PLAN CURVES

RWS (ft, NAVD88)	Total Capacity (per NHC CFD model, 2020)					
	Spillway Only cfs	Power Intake Only cfs	One culvert - no power cfs	One culvert with power cfs	Two culverts cfs	Two culverts + spillway cfs
3801.7	30,402	2,850	3,786	6,636	7,572	37,974
3800.7	27,680	2,850	3,740	6,590	7,480	35,160
3799.7	25,045	2,850	3,694	6,544	7,388	32,433
3798.7	22,500	2,850	3,647	6,497	7,294	29,794
3797.2	20,046	2,850	3,599	6,449	7,198	27,244
3796.7	17,690	2,850	3,550	6,400	7,100	24,790
3795.7	15,433	2,850	3,501	6,351	7,002	22,435
3794.7	13,282	2,850	3,451	6,301	6,902	20,184
3793.7	11,241	2,850	2,915	5,765	5,830	17,071
3792.7	9,265	2,850	2,868	5,718	5,736	15,001
3791.7	7,433	2,850	2,820	5,670	5,640	13,073
3790.7	5,752	2,850	2,772	5,622	5,544	11,296
3789.7	4,233	2,850	2,723	5,573	5,446	9,679
3788.7	2,887	2,805	2,674	5,479	5,348	8,235
3787.7	1,733	2,531	2,623	5,154	5,246	6,979
3786.7	801	2,269	2,572	4,841	5,144	5,945
3785.7	153	2,020	2,520	4,540	5,040	5,193
3785.2	-	1,784	2,494	4,278	4,988	4,988
3784.7	-	1,561	2,467	4,028	4,934	4,934
3783.7	-	1,351	2,414	3,765	4,828	4,828
3782.7	-	1,155	2,359	3,514	4,718	4,718
3781.7	-	973	2,303	3,276	4,606	4,606
3780.7	-	805	2,240	3,045	4,480	4,480
3779.7	-	651	2,187	2,838	4,374	4,374
3778.7	-	512	2,128	2,640	4,256	4,256
3777.7	-	388	2,066	2,454	4,132	4,132
3776.7	-	279	2,003	2,282	4,006	4,006
3775.7	-	187	1,939	2,126	3,878	3,878
3774.7	-	111	1,872	1,983	3,744	3,744
3773.7	-	54	1,803	1,857	3,606	3,606
3772.7	-	15	1,731	1,746	3,462	3,462
3771.7	-	-	1,657	1,657	3,314	3,314
3770.7	-	-	1,578	1,578	3,156	3,156
3769.7	-	-	1,496	1,496	2,992	2,992
3768.7	-	-	1,409	1,409	2,818	2,818
3767.7	-	-	1,316	1,316	2,632	2,632
3766.7	-	-	1,135	1,135	2,270	2,270
3765.7	-	-	1,098	1,098	2,196	2,196
3764.7	-	-	868	868	1,736	1,736
3763.7	-	-	735	735	1,470	1,470
3762.7	-	-	609	609	1,218	1,218
3761.7	-	-	491	491	982	982
3760.7	-	-	382	382	764	764
3760.2	-	-	331	331	662	662
3760.0	-	-	312	312	624	624
3759.7	-	-	283	283	566	566
3758.7	-	-	194	194	388	388
3757.7	-	-	117	117	234	234
3756.7	-	-	54	54	108	108
3755.7	-	-	10	10	20	20
3755.2	-	-	-	-	-	-

Notes: *Power intake rating curve provided by KP, Ref. VA103-640-01

J.C. BOYLE POST-DRAWDOWN RESERVOIR WATER SURFACE ELEVATIONS AT THE EMBANKMENT

Discharge (cfs)	Embankment Reservoir Water Surface Elevation (ft., NAVD88)	Modelling Method
1,700	3767.9	Hec-RAS 2D
1,900	3768.0	
2,000	3768.1	
2,100	3768.1	
2,200	3768.2	
2,300	3768.4	
2,400	3768.7	
2,500	3769.7	
2,800	3769.8	
2,900	3770.2	
3,000	3771.1	
3,100	3772.5	
3,400	3776.0	
3,900	3777.8	
4,100	3778.5	
4,200	3779.4	
4,300	3780.1	
4,400	3781.7	
4,600	3784.6	
4,900	3787.4	
6,300	3788.0	
6,800	3788.4	
7,300	3789.1	
8,000	3790.1	
9,400	3790.4	
9,700	3790.6	
9,900	3791.0	
10,500	3791.4	
10,900	3793.2	
13,600	3793.5	
14,300	3793.7	
16,200	3794.1	Flow 3D
17,300	3794.4	
18,800	3794.7	